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Waterways and Harbors Division

PROCEEDINGS OF THE
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OF CIVIL ENGINEERS

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CONTENTS

May, 1961

Papers

Page

Design and Stability Considerations for Unique Pier
by James Michalos and David P. Billington 1

Economics of River Bank Stabilization
by Charles Senour 17

Cathodic Protection of Marine Terminal Facilities
by Joseph Milano 27

Hurricane Winds for Design Along the New England Coast
by C. S. Gilman and V. A. Myers. 45

Pneumatic Breakwaters to Protect Dredges
by James L. Green 67

(over)

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Note.—Part 2 of this Journal is the 1961-16 Newsletter of the Waterways and Harbors Division.

The three preceding issues of this Journal are dated September 1960, November 1960, and February 1961.

Page

Groins on the Shores of the Great Lakes
by Charles E. Lee 89

Deep Water Wave Generations by Moving Wind Systems
by Basil W. Wilson 113

DISCUSSION

Breakwaters in the Hawaiian Islands, by Robert Q. Palmer. (June, 1960. Prior discussion: November, 1960, February, 1961. Discussion closed.)
by M. H. Marty and E. E. Chapus 145

Prediction of Maximum Practical Berth Occupancy, by Thomas J. Fratar, Alvin S. Goodman, and Austin E. Brant, Jr. (June, 1960. Prior discussion: November, 1960. Discussion closed.)
by Thomas J. Fratar, Alvin S. Goodman, and Austin E. Brant, Jr. (closure). 149

Tetrapods and Other Precast Blocks for Breakwaters, by P. Danel, E. Chapus, and R. Dhaille. (September, 1960. Prior discussion: February, 1961. Discussion closed.)
by T. P. Anantram 151
by Shri M. G. Hiranandani, C. V. Gole and Y. D. Pendse 151

Shark River Inlet Sand By-Passing Project, by W. Mack Angus. (September, 1960. Prior discussion: None. Discussion closed.)
by Per Bruun. 153

Pneumatic Barrier, by Ian Larsen. (September, 1960. Prior discussion: None. Discussion closed.)
by T. M. Dick 159

Studies of a Channel Through Padre Island, Texas, by E. A. Hansen. (September, 1960. Prior discussion: None. Discussion closed.)
by Per Bruun. 161

Design of Inlets for Texas Coastal Fisheries, by H. C. Innis and H. P. Carothers. (September, 1960. Prior discussion: None. Discussion closed.)
by F. Gerritsen 165

Russian River Channel Works, by Israel H. Steinberg. (November, 1960. Prior discussion: None. Discussion closed.)
by R. Robinson Rowe. 169
by Otto C. Von Seggern 170

(over)

Page

Sacramento River Deep Water Channel: Functional Planning, by Amalio Gomez. (November, 1960. Prior discussion: None. Discussion closed.)
by Blair T. Bower 177

Design Consideration for California Marinas, by James W. Durham. (November, 1960. Prior discussion: None. Discussion closed.)
by J. W. B. Blackman 179

Great Volga Waterway, by Otakar W. Kabelac. (February, 1961. Prior discussion: None. Discussion closes June 1, 1961.)
by Roy F. Bessey 185

Errata 189

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

DESIGN AND STABILITY CONSIDERATIONS FOR UNIQUE PIER

By James Michalos,[1] F. ASCE, and David P. Billington,[2] A. M. ASCE

SYNOPSIS

Design and stability of Pier 40 in New York City are described. Particular attention is paid to the method of analysis used for studying the stability of the pier when subjected to lateral impact.

INTRODUCTION

This paper describes the design and stability consideration of a unique cargo and passenger pier on the Manhattan shore of the Hudson River, which is capable of handling four ships simultaneously. Constructed under the direction of the Department of Marine and Aviation of the City of New York for lease to the Holland-America Line, the terminal, to be known officially as Pier 40, replaces five old finger piers which, although providing ten berths, had completely inadequate shed and berthing areas.

Special features include provision for simultaneous, but completely separated, large-scale cargo and passenger operations; on-site handling and parking of trucks and trailers, completely removed from cargo areas; pick-up and discharge of passengers by private automobiles traversing an interior peripheral roadway on the passenger level; streamlined flow of cargo and passengers between ship and shore; and provision for automobile parking facili-

Note.—Discussion open until October 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 2, May, 1961.

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[2] Proj. Mgr., Roberts and Schaefer Co., New York, N. Y.

ties alongside the passenger deck and on the roof. Cargo operations are restricted to the main deck, and passenger operations are all concentrated on the second deck.

GENERAL DESCRIPTION

Fig. 1 is a photograph of an artist's rendering of the pier. It is roughly 800 ft square, with a 140-ft finger projection at the southwest corner, and has wider than usual berths (140 ft) at the upstream and downstream faces. The Burtoning system for handling cargo is evident along the periphery of the roof deck. An unusually wide (20 ft) apron is provided around the periphery on the main deck.

The cargo deck is designed for a live load of 600 psf, and the truck court is designed for a live load of 300 psf or H20-S16 truck-trailer loading. The clear height to the second deck varies from 11 ft to 14 ft. In general, trucks are restricted to the truck court which is tail-gate height below the cargo deck. Ramps between truck court and cargo deck are provided, however, to enable the driving of trucks on the cargo deck when necessary.

The second (passenger) deck is shown schematically in Fig. 2. It is designed for a live load of 100 psf. Automobiles and baggage trucks entering the terminal at street level will drive up a ramp to a peripheral driveway provided for convenient discharge or loading of passengers and visitors. Room will be available for storing automobiles of departing passengers. Ramps provide for driving directly to the roof deck where there is space for the commercial parking of approximately 800 cars. That deck is designed for a live load of 75 psf.

The North American operating offices of the Holland-America Line are located on the second deck, along the inshore face, and operating offices for the terminal are located below, on the first deck. Waiting room and eating areas are also located on the second deck.

The terminal is constructed in three phases: (a) substructure, including demolition of old piers; (b) superstructure, consisting of all structural work above the main deck; and (c) general construction, consisting of all architectural, mechanical, and electrical work. The total estimated construction cost is approximately $17,700,000, which is broken up, roughly, into $8,500,000 for substructure, $4,000,000 for superstructure, and $5,200,000 for general construction.

SUBSTRUCTURE

The structure is supported on steel bearing piles (14 BP 89) driven to rock and designed for a total compressive force of 105 tons. Before it was decided to use steel piles, several other systems, including wood piles, cast-in-place concrete pipe piles, and precast concrete piles, were investigated. Structural steel piles were chosen primarily because of their immediate availability at the lowest competitive price. New York City ordered these piles under a separate contract, previous to award of the general foundation contract, for a price less than 6 cents per pound delivered to the site.

When steel piles are used, corrosion protection must be considered. In New York harbor a variety of experience with steel piles in water is available,

FIG. 1.—ARTIST'S RENDERING

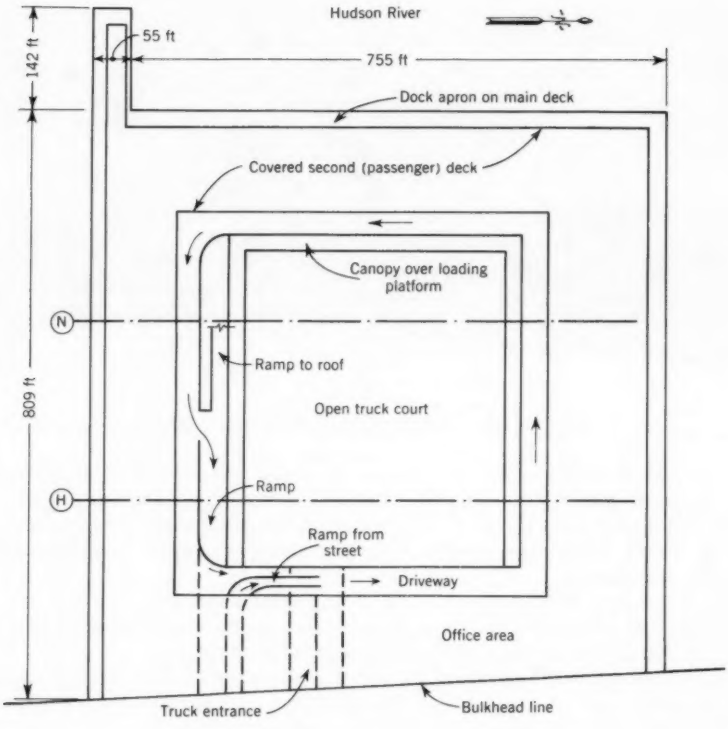

FIG. 2.—SCHEMATIC PLAN

but it is not at all consistent. In some areas structural steel piling has apparently been satisfactory, but in other areas a significant loss of steel area has been noted after a few years. For this project it was considered essential to provide against electrolytic corrosion by means of a cathodic protection system. In the splash zone between low water and high water a three-layer bituminous coating was specified to prevent significant corrosion resulting from alternate exposure to air and water.

Test borings were made at various locations on the site. Depth from water surface to mud line varied from 20 ft to 30 ft alongside the existing piers which were to be demolished. Directly beneath these piers the depth varied from 2 ft to 10 ft. The existing mud line was not disturbed except around the periphery of the new terminal, where it was dredged to 35 ft.

Below the mud line the silt gradually increases in density to a depth approximately 80 ft below mean low water elevation, at which depth generally a layer of sand begins and extends to rock which is found at depths varying from

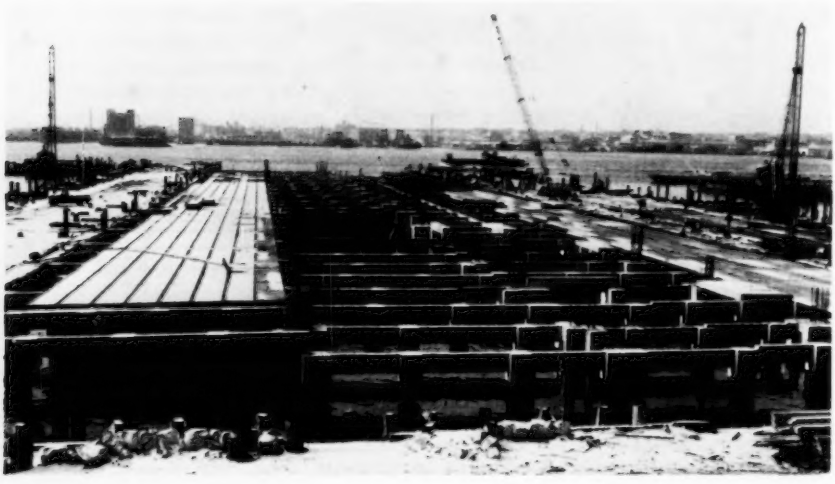

FIG. 3.—MAIN DECK UNDER CONSTRUCTION

75 ft to 160 ft below mean low water. In areas where rock is roughly 75 ft to 85 ft below mean low water there is no layer of sand. During pile driving operations it was noted that in these areas the steel piles dropped directly to rock under their own weight, thus indicating virtually complete lack of frictional resistance.

Pile bents are spaced roughly 20 ft on centers. As shown in Fig. 3, concrete pile caps are cast integrally with continuous beams, generally spanning in a direction perpendicular to the periphery of the structure.

The narrow extension piece at the southwest corner of the pier is supported on steel piles, as is the rest of the structure, except that batter piles are provided around its periphery. Unlike the main structure, the extension piece is narrow and requires few piles for the vertical loads since it has only a one-story shed. Batter piles were introduced to provide lateral stability for this portion of the structure.

Several types of fendering systems were investigated for the 2,600 lineal ft of docking space. Originally, a system utilizing rubber blocks as bumpers was designed; however, the cost was too high. Because this was true of other methods studied it was decided to provide nothing more than a simple system of timber fendering. This consists of white oak piles driven about 45 ft into the mud and connected to the peripheral concrete fender girder by means of wooden chocks and whales. Special buffer dolphins are provided at the three outshore corners of the terminal. All of the piles and timbers used in these vulnerable areas are greenheart, and hollow rubber tubes are provided to help absorb impact energy.

Prestressed panels span between pile bents. Concrete keys were cast between adjacent panels to tie them together transversely, and concrete was also cast between panel ends over the beams. Prestressing strands were left extending from the ends of the panels into the cast concrete, as shown in Fig. 4, so as to provide a lateral tie over the beams. A bituminous wearing surface is used over the panels to provide a smooth even finish for the main deck.

Construction of the substructure and main deck proceeded in the following manner:

1. The sheds of the five old piers were demolished.
2. Piling was shipped, loaded onto old Pier 37, spliced to length, and coated.
3. Piling was driven in the areas between existing piers by water rigs and through the old pier decks by land rigs.
4. Beam forms were erected over piles located between the old piers, and the beams were cast from these piers.
5. Floor panels, shipped by truck from a casting plant about 10 miles away, were erected by means of cranes placed on the existing piers. After several rows of panels were placed, connecting joints were cast and, when they had acquired design strength, the cranes were able to move onto the panels and erect adjacent rows.
6. When sufficient panels had been erected to permit working room, the old pier substructures were demolished and concrete beams cast over the previously driven piles.

Nearly half of the cast-in-place concrete was placed in winter. The contractor was permitted to cast when the temperature was above 20°F, but he was required to maintain a 50°F concrete temperature for three days after casting. This was achieved without internal heat by using insulation on the sides of the forms and by covering the top of the concrete with salt hay and tarpaulins.

The contract provided for load tests to be performed on the precast prestressed panels when deemed necessary. Several times during the job 28-day cylinder strengths were below the contract requirement, generally 5,000 psi. Corresponding panels were, therefore, designated for testing, as were certain panels cast and cured under condition of low temperature. Loading was applied gradually up to two times design load by means of two 100-ton hydraulic rams spaced roughly at the quarter points of the spans (the spans varied from 20 ft to 25 ft). Small cracks appeared which were evenly distributed in the central half of the panel but they disappeared on release of the load. The performance of these panels was good enough to permit the contractor to utilize them in the construction.

Two panels were tested to destruction. They withstood a superimposed load of 3.5 times design live load before failure by yielding of the strands and subsequent crushing of the concrete. Based on an ultimate strength analysis, this indicates a compressive strength of concrete in the neighborhood of 6,000 psi whereas the 28-day cylinder strengths were under 4,000

FIG. 4.—PLACING OF DECK PANELS

psi. There was apparently no slip of the strands even though the failure crack was only 4 1/2 ft from the end of the panel.

SUPERSTRUCTURE

A complete design of the superstructure was prepared based on a prestressed concrete framing system. Prior to asking for bids it was concluded

that, as a result of changed circumstances regarding availability and prices, structural steel framing might be more economical. For that reason the possibility of submitting an alternate bid based on structural steel framing was written into the specifications. It was required that any such bid be based on a fireproof design acceptable to the city. The low bid in prestressed concrete was 7% less than the low bid in structural steel. In addition, it was approximately 7% below the engineers' estimate of $4,268,000.

Fig. 5 shows various stages of completion of the superstructure. Precast deck panels are stacked in the truck court, ready for use.

Typical framing for the concrete superstructure is shown in Fig. 6. On the second deck the beams and girders act as simple spans under dead load but, with the floor slab in place, act as continuous members under live load.

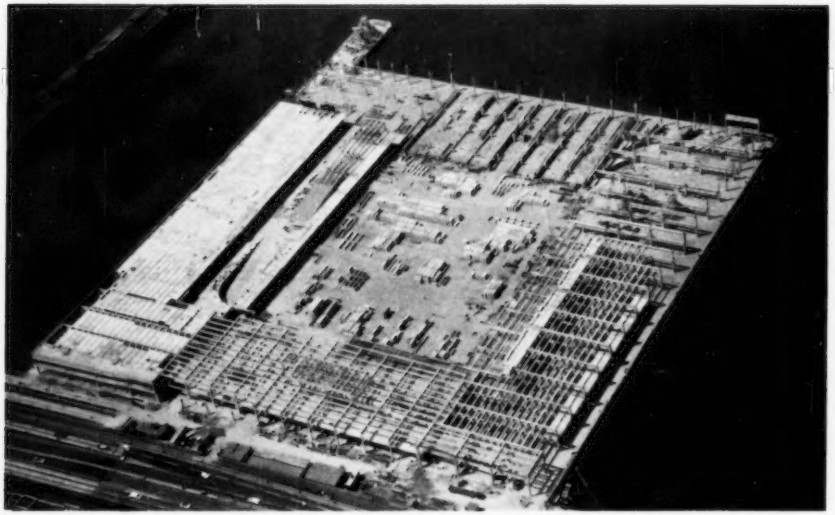

FIG. 5.—AERIAL PHOTOGRAPH DURING CONSTRUCTION

Certain functional requirements resulted in some of the second deck girders being highly stressed. The top elevation of the second deck was fixed by the operational requirement for passenger gangplanks from the ships. The clearance from the main deck to the under side of the second deck girders was fixed by cargo handling requirements. The girders, spaced 40 ft to 50 ft on centers, carry heavy loads; and, to reduce maximum stresses as much as practicable under the circumstances, they were designed with single or double cantilever ends. To reduce stresses further, shoring and two stages of prestressing were used in the following manner:

1. Upon reaching a specified strength (4,000 psi or 5,000 psi, depending on design strength) a typical double cantilever was post-tensioned with about two-thirds of its total prestressing force.

2. The girder was then erected and shoring placed directly below the location of the roof column.

3. The remainder of the precast superstructure was erected with the shoring in place so as to prevent any deflection of the girder under the roof column.

4. With all the precast elements of the second deck and roof deck in place, the remaining prestressing force was applied to the girder and the shoring removed.

5. The second deck slab was then cast in place.

By this method prestressing is provided for the entire dead load and live load on the structure, but initial overstressing is avoided.

The post-tensioned girders were precast at the site, adjacent to the columns which support them. The prestressed concrete beams were manufactured in a casting yard and trucked approximately 10 miles to the site where cranes erected them directly from the trucks. A cast-in-place slab, 5 in. thick, ties the entire second deck framework together.

The post-tensioned girders and the pre-tensioned beams on the roof deck are generally not continuous. The channel slabs, of lightweight reinforced con-

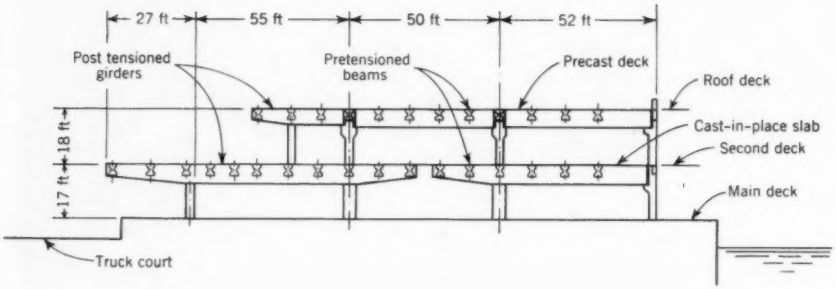

FIG. 6.—TYPICAL SUPERSTRUCTURE FRAMING

crete, were cast in molds at the site. They were made continuous and composite with the beams by means of a cast-in-place concrete strip over the beams. The beams at the column line are made continuous to act as a lateral tie for the entire structure.

Generally, prestressed concrete members were designed using a 28-day concrete strength of 5,000 psi, but the most heavily stressed members were designed using 6,000-lb concrete. Increased concrete strength was specified in order to avoid increased sizes.

STABILITY ANALYSIS

A typical analysis of the stability of the pier included all piles between expansion joints on column lines H and N (Fig. 2) with a ship striking either the north or south end of this portion of the terminal. Virtually all the 889 piles involved are oriented so that bending due to ship impact is about the minor principal axis of the pile cross section.

It is assumed that a 36,000-ton ship, with a velocity component of 0.55 fps perpendicular to the face of the pier, strikes along the center of gravity of the piles. It is further assumed that the collision is elastic, that is, no energy is dissipated through damage to ship or pier. Furthermore, energy losses resulting from elastic deformation of the ship's hull, from rolling or yawing of the ship, or from piling up of water are neglected. This assumption is conservative. It has been estimated[3] that anywhere from 25% to 90% of the kinetic energy is delivered to a pier, 40% being considered a fair estimate for the type of ship here under consideration.

From considerations of conservation of momentum and kinetic energy,

$$m_1 v_1 + m_2 v_2 = m_1 v_1' + m_2 v_2' \quad \ldots\ldots\ldots\ldots \quad (1)$$

and

$$\frac{1}{2} m_1 v_1^2 + \frac{1}{2} m_2 v_2^2 = \frac{1}{2} m_1 v_1'^2 + \frac{1}{2} m_2 v_2'^2 \quad \ldots\ldots\ldots \quad (2)$$

Eliminating v_1' and solving for v_2',

in which m_1 is the mass of ship, m_2 denotes the mass of pier, v_1 refers to the velocity of ship before impact, v_1' is the velocity of ship after impact, v_2 describes the velocity of pier before impact, and v_2' denotes the velocity of pier after impact.

$$v_2' = \frac{2 m_1}{m_1 + m_2} v_1 + \frac{m_2 - m_1}{m_1 + m_2} v_2 \quad \ldots\ldots\ldots\ldots \quad (3a)$$

However, since v_2 is equal to zero, Eq. 3 reduces to

$$v_2' = \frac{2m_1}{m_1 + m_2} v_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3b)$$

If it is assumed that approximately two-thirds of the design live load will be present (average load per pile equal to 175 kips), then

$$m_2 = \frac{175 \times 889}{32.2} = 4850 \text{ kip sec}^2 \text{ per ft}$$

Now since

$$m_1 = \frac{72{,}000}{32.2} = 2224 \text{ kip sec}^2 \text{ per ft}$$

we obtain from Eq. 3(b)

$$v_2' = \frac{2\ (2224)}{2224 + 4850}\ (0.55) = 0.346 \text{ fps}$$

For a single-degree-of-freedom system which has been subjected to an initial velocity v_2', it can be shown that, assuming no damping, the maximum acceleration is

$$a = \omega v_2' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

and the maximum displacement is

$$\delta = \frac{v_2'}{\omega} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

---

3 "Docking Fenders Key to Pier Protection," by Louis A. Volse, Engineering News Record, May 8, 1958.

in which ω, the so-called circular velocity, is

$$\omega = \sqrt{\frac{k}{m_2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

in which k is the spring constant, or force necessary to displace the system a unit distance from its equilibrium position.

The piles are rigidly fixed by the girders into which they frame, and the girders are of such dimensions that they can be considered to have infinite end rotational stiffness relative to the piles. Since the latter can also be considered fixed against rotation at some depth below the mud line, the spring constant per pile would be, if no vertical load were present,

$$\frac{F}{\delta_o} = \frac{12\,E\,I}{L^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

in which δ_o is the relative horizontal displacement, with no rotation, of the top of the pile due to a horizontal load F applied at the top; E is the modulus of elasticity; I is the moment of inertia of the pile cross-section about the axis of bending; and L is the distance between the fixed top and the fixed bottom of the pile.

Since a vertical load is present, the expression for k must be modified because a vertical load will increase the lateral deflection. A pile subjected to horizontal and vertical loads is shown in its deflected position in Fig. 6. As an increasingly larger load P is applied, the deflections are magnified over those that would occur if no vertical load were present. Finally, when the critical load, P_{cr}, is attained, the deflections increase without limit.

The magnification of deflection due to vertical load can be readily determined by substituting an equivalent pin-end column of length L and initial deflection δ_o. Then, assuming the deflected shape as a half-sine curve, successive increments of deflection due to P can be computed as follows:

$$\delta_1 = \frac{P\,\delta_o\,L^2}{\pi^2\,E\,I} = \frac{P}{P_{cr}}\,\delta_o \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8a)$$

$$\delta_2 = \frac{\delta_1}{\delta_o}\,\delta_1 = \frac{P}{P_{cr}}\,\delta_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8b)$$

$$\delta_3 = \frac{\delta_1}{\delta_o}\,\delta_2 = \frac{P}{P_{cr}}\,\delta_2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8c)$$

and so on.
But

$$\delta_{max} = \delta_o + \delta_1 + \delta_2 + \delta_3 + \text{---} \quad \ldots\ldots\ldots\ldots\ldots \quad (9a)$$

and substituting from Eqs. 8

$$\delta_{max} = \delta_o + \frac{P}{P_{cr}}\left(\delta_o + \delta_1 + \delta_2 + \delta_3 + \text{---}\right) \quad \ldots\ldots\ldots \quad (9b)$$

or

$$\delta_{max} = \delta_o + \frac{P}{P_{cr}}\,\delta_{max} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9c)$$

Therefore

$$\delta_{max} = \frac{\delta_o}{1 - \frac{P}{P_{cr}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

This expression is given by S. Timoshenko[4] but it was felt the preceding derivation would be of interest as clearly showing the basis for the expression and the amount of approximation involved.

The spring constant k, including the effect of vertical load, is, by definition,

$$k = \frac{F}{\delta_{max}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

and, substituting from Eq. 10,

$$k = \frac{F}{\delta_o}\left(1 - \frac{P}{P_{cr}}\right) \quad \ldots\ldots\ldots\ldots\ldots \quad (12)$$

which, from Eq. 7, becomes

$$k = \frac{12\ E\ I}{L^3}\left(1 - \frac{P}{P_{cr}}\right) \quad \ldots\ldots\ldots\ldots\ldots \quad (13)$$

For the piles under consideration, take I=326 in.^{4}; L=512 in.; $\frac{12\ E\ I}{L^3}$ = 0.88 kips per in. for one pile; = 780 kips per in. for 889 piles; and P = 175 kips. (The values for L and $12\ E\ I/L^3$ were actually obtained on the basis of a field test described subsequently.) The critical buckling load is then

$$P_{cr} = \frac{\pi^2\ E\ I}{L^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

$$P_{cr} = \frac{\pi^2 \left(30 \times 10^3\right)\ 326}{(512)^2}$$

$$P_{cr} = 367 \text{ kips}$$

and, from Eq. 13, the spring constant is

$$k = \frac{12\ E\ I}{L^3}\left(1 - \frac{175}{367}\right) = 6.30\ \frac{E\ I}{L^3}$$

or

$$k = 410 \text{ kips per in. for 889 piles.}$$

Then, from Eq. 6,

$$\omega = \sqrt{\frac{410\ (12)}{4850}} = 1.01 \text{ radians per sec}$$

and from Eqs. 4 and 5 respectively,

$$a = -1.01\ (0.346) = -0.349 \text{ ft per sec}^2$$

$$\delta = \frac{0.346}{1.01} = 0.343 \text{ ft} = 4.11 \text{ in.}$$

---

4 "Theory of Elastic Stability," by S. Timoshenko, McGraw-Hill Book Co., New York, 1936, p. 29.

The force delivered to the pier is

$$F = m_2 a \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (15)$$

$$F = 4850\ (0.349)$$

$$F = 1690 \text{ kips}$$

or

$$\frac{1690}{889} = 1.90 \text{ kips per pile}$$

The maximum bending moment in the pile (see Fig. 6) is then

$$M_{max} = \frac{P\,\delta}{2} + \frac{F\,L}{2}$$

$$M_{max} = 175\ (2.06) + 1.90\ (256)$$

$$M_{max} = 847 \text{ in. kips}$$

and the maximum fiber stress is

$$\sigma = \frac{P}{A} + \frac{M}{I/c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (16)$$

$$\sigma = \frac{175}{26.19} + \frac{847}{44.4}$$

$$\sigma = 25.8 \text{ ksi}$$

In the preceding analysis no allowance was made for the fender pile system or for the kinetic energy dissipated. These two factors will result in much lower stresses. It is assumed that ten timber fender piles will participate in reducing the impact. The spring constant for each of these piles is estimated at 7 kips per in. deflection, or 70 kips for ten piles.

For 889 steel piles and ten fender piles acting in series the combined spring constant can be determined as follows:

$$\frac{1}{k} = \frac{1}{410} + \frac{1}{70}$$

$$k = 60 \text{ kips per in.}$$

Then, proceeding as before,

$$\omega = 0.385 \text{ radian per sec}$$

$$a = 0.133 \text{ ft per sec}^2$$

$$F = 4850\ (0.133) = 645 \text{ kips}$$

and

$$\delta = \frac{645}{410} = 1.57 \text{ in.}$$

With F equal to 645/889, or 0.73 kip per pile, and $\delta/2$ equal to 0.79 in., the maximum bending moment is

$$175\ (0.79) + 0.73\ (256) = 325 \text{ in. kips}$$

and the maximum stress is

$$\frac{175}{26.19} + \frac{325}{44.4} = 14.0 \text{ ksi}$$

Computations indicate that, under the unusual condition of full impact, the fender piles may rupture. This is not considered serious since they will very nearly have performed their function and they can readily and cheaply be replaced.

The reduced lateral stiffness (spring constant with effect of axial load included) can be determined precisely for members with or without haunched ends.[5] The charts presented by J. Michalos consist of plots of end rotation stiffness factors and carry-over factors versus a parameter k m l, in which k is equal to $\sqrt{\frac{P}{E\,I}}$ and m is equal to 1 for a prismatic member. Then, for the piles under consideration

$$k\,m\,L = L\sqrt{\frac{P}{E\,I}} = 2.16 \quad \ldots\ldots\ldots\ldots\ldots \quad (17)$$

From the charts, the stiffness factor is 3.33 and the carry-over factor is 0.65. The fixed-end moments due to unit translation of one end relative to the other are then

$$(1 + 0.65)\ 3.33\ \frac{E\,I}{L^2} = 5.50\ \frac{E\,I}{L^2}$$

The end moments and the vertical load of 175 kips are shown in Fig. 7. The value of the spring constant k is determined from the requirements of equilibrium as follows:

$$k\,L = (2)\ \frac{5.50\ E\,I}{L^2} - 175\ (1)$$

$$= \frac{11.00\ E\,I}{L^2} - 175\ \frac{E\,I\,L^2}{E\,I\,L^2}$$

$$= \frac{11.00\ E\,I}{L^2} - \frac{175\ E\,I\ (512)^2}{30{,}000\ (326)\ L^2}$$

and

$$k = 6.30\ \frac{E\,I}{L^3} \quad (18)$$

which is identical to the result obtained earlier from Eq. 13. The same result could also be obtained using B. W. James' formula for prismatic members.[6]

In order to substantiate the design assumptions with respect to behavior under collision loads, Ralph B. Peck, F. ASCE, was retained by the structural designers to study the soil conditions and make a report. In that report it was concluded that the pier will be stable under the impact of a 36,000-ton ship

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[5] "Theory of Structural Analysis and Design," by James Michalos, Ronald Press Co., New York, 1958, pp. 446-453 and Appendix, Figs. 1 to 25.

[6] "Principal Effects of Axial Load on Moment-Distribution Analysis of Rigid Structures," by Benjamin Wylie James, NACA Tech. Note No. 534, July, 1935, pp. 25-26.

striking the pier with a velocity of 0.55 fps. Particular attention was paid to the effect of variable depth to the mud line.

FOUNDATION INVESTIGATIONS

Investigations were made in the field in order to determine the following design information:

1. Depth below mud line at which piles can be considered fixed.
2. The restraint on the pile top offered by the concrete pile cap.
3. The relationship between lateral load and lateral movement of the pile tops.

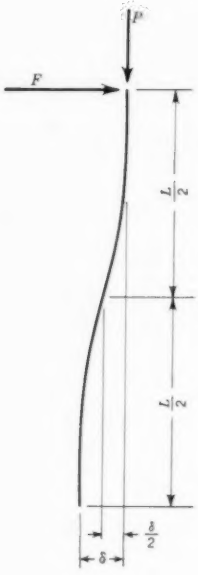

FIG. 7.—DISPLACED POSITION OF A PILE

FIG. 8.—UNIT DISPLACEMENT OF PILE

In order to help determine the depth at which the piles could be considered fixed, a Wilson tiltmeter was used on one of the piles. On the basis of readings made when a lateral load was applied to the pile, it was concluded that the point of fixity for this pile was approximately 17 ft below the mud line.

More than 3,000 piles were used. To get an economical and quick comparison of soil stiffness in different locations, a series of shear vane tests were made. These tests indicated a uniformity of lateral resistance throughout the area.

The fixity of the pile tops was tested by pushing a line of piles, over which the pile caps and connecting beams had been cast, by means of a 50-ton hy-

draulic ram reacting against one of the old piers before demolition. The movement of the pile line was measured at each end and recorded for each increment of load. The first test was made with a movement that might be expected under design conditions, and the second test was made with a movement almost two and one-half times as large. For test no. 1: P = 22.8 kips for sixteen piles; and δ = 1.62 in.

For fixed top and bottom:

$$P = \frac{12\,E\,I\,\delta}{L^3} \qquad (19)$$

$$L^3 = \frac{12\,\left(30 \times 10^3\right)(326)\,(1.62)}{22.8/16}$$

$$L = 512 \text{ in.} = 42.7 \text{ ft}$$

For test no. 2: P = 60.3 kips for sixteen piles; δ = 3.96 in; and L = 41.6 ft. Soundings show that the mud line is approximately 20 ft below the mean low water line, and the tiltmeter, as previously stated, indicated 17 ft to point of fixity. The pile cap soffits are an average of 7.5 ft above mean low water. The total length to bottom fixity would then be approximately

$$L = 20 + 7.5 + 17 = 44.5 \text{ ft}$$

This compares fairly well with the results of the push tests. It seems reasonable to assume that the pile tops are fixed against rotation and that lateral movements are proportional to load.

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ECONOMICS OF RIVER BANK STABILIZATION

By Charles Senour,[1] F. ASCE

SYNOPSIS

Determination of costs pertaining to stabilization of the caving banks of an alluvial river is not particularly difficult, but realistic appraisal of resultant benefits presents some interesting problems in both the physical and accounting fields. In the latter, comparison of total benefits during the life of a bank stabilization project with total costs during the same period is suggested as possibly being a better criterion of economic merit than the ratio between annual "equivalent" benefits and annual costs if interest rates pertaining to benefits differ materially from those pertaining to costs. In the physical field, gradually changing patterns of land use, and growth in population and industrial development challenge the adequacy of losses experienced in past decades as a criterion of future benefits from prevention of continued bank recession. The existing or foreseeable engineering construction, (particularly reservoirs) offer the same challenge. The role of accretion in restoration of revenues is somewhat controversial.

The use of estimated future bank lines as a basis for computing benefits eliminates certain of the objectionable aspects of sole reliance on historical bank recession and damages. The criterion of benefits from stabilization and correlation between river stage and rate of bank recession provides a means of adjusting estimates of future land loss to reflect the influence of existing or authorized reservoir construction. Comparison of aerial photographs of

Note.—Discussion open until October 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 2, May, 1961.

[1] Assoc. Tippetts-Abbet-McCarthy-Stratton, New York, N. Y.

selected localities taken at intervals during a period of years affords a basis for appraisal of the compensating effects of accretion.

TYPES OF LOSSES PREVENTED BY STABILIZATION

The primary benefit resultant from stabilizing the caving banks of a river is prevention of loss of land and improvements. There is an important distinction to be drawn between the two types of loss. A barn that caves into the river can be immediately replaced. A threatened barn can often be moved away from the bank before the river engulfs it. A structure continues in service or, by replacement, immediately goes back into service. But the revenue from an acre of land destroyed in any one year is also lost for each succeeding year thereafter. The construction of bank-stabilization works, with a useful life of 50 yr, would permit the realization of annual revenue from the acreage which, without the protection; would cave into the river the first year of the project's life; would make possible for 49 yr, annual revenue from the acreage that in absence of the protection would cave into the river the second year; and so on for 50 yr. The total of benefits during the 50-yr life of the project, in terms of net annual returns from the protected acreage, would be 1,275 times the average number of acres which without protection, would be lost annually by caving, multiplied by the average annual net return per acre.

VARIOUS BASES OF COMPARISON BETWEEN BENEFITS AND COSTS

In order to appraise the economic worth of the project, benefits must be compared with costs. The latter normally consist of amortization of investment, maintenance, and operation, and customarily are reported on an annual basis. The simplest comparison between benefits and costs of a stabilization project with a 50-yr life, would be to compare total costs during the 50-yr life of the project with 1,275 times the average annual loss of net revenue prevented by arresting bank recession.

The benefits, however, do not accrue at a uniform rate. Problems of compound interest are involved. The prescribed procedure is to treat the benefits as an increasing annuity, to compute the present value of the increasing annuity, and by applying to the present value the amortization factor, to arrive at what is termed the average annual "equivalent" benefit. The "present value" or "present worth" of an increasing annuity is the capital sum which, if placed at interest, at a given rate, will amount at the end of the life of the annuity to the sum of the compound amounts of all the annuity payments. The average annual "equivalent" benefit is the annual rental of a constant annuity that, with a life equal to that of the increasing annuity and with the same rate of interest as the increasing annuity, would yield compound amounts whose sum would equal the sum of the compound amounts of the increasing annuity.

Usually, bank stabilization projects of significant magnitude are undertaken by the federal government. Interest rates vary from time to time, but a generally accepted standard rate for the federal government's borrowing is 2 1/2% per annum. If a bank stabilization project having a useful life of 50 yr, prevented bank caving which each year was destroying land from which the

net revenue was $3,000, the benefits would take the form of an increasing annuity of $3,000. If the benefits accrued directly to the government, the annuity's interest rate would presumably be 2 1/2% and the annual average "equivalent" benefit, approximately $61,000. The project would have a benefit-cost ratio in excess of unity if annual charges for maintenance and for amortization of investment were less than $61,000.

Ordinarily the benefits from a stabilization project accrue to private landowners or associations thereof, rather than directly to the federal government. Interest rates vary for private borrowing also, but in economic analyses it is more or less standard practice to assume in connection with non-federal funds, an interest rate of 4% per annum. It has been decreed that the average annual "equivalent" benefit, if it accrues to non-federal agencies, be computed on a 4% basis. Computed on the basis of 4%, the average annual "equivalent" benefit would be approximately $53,500 instead of $61,000. The actual physical benefits, and the annual costs, assuming the same basis of maintenance in each instance, would be unchanged. But with annual charges of $55,000, the project which would show a benefit-cost ratio of approximately 1.1, with the government as the direct beneficiary, apparently would fail of economic justification (benefit-cost ratio 0.97) if a non-federal agency were the beneficiary, and the benefits earned 4% instead of only 2 1/2% interest.

Perhaps the philosophy underlying this seeming anomaly is that since non-federal borrowing is at 4%, non-federal earning is also 4%, so that the average annual "equivalent" benefit of $53,500 represents the annual subsidy that would be required to compensate owners for the loss of land, inasmuch as they could earn 4% interest on it, compounded. This is of small comfort to the landowners, who know there is not the remotest chance of such a subsidy being granted.

It is suggested that the true effect of a bank-stabilization project on the economy might better be gaged by comparing total costs of its life with total earnings; in each case using the applicable interest rates, compounded as a means of compensating for deferment of benefits. Annual charges of $55,000, financed from loans at 2 1/2% might aggregate approximately $5,400,000 in 50 yr. Annual benefits, increasing at the annual rate of $3,000, computed as an increasing annuity at 4%, would increase in 50 yr to approximately $8,150,000.

PHYSICAL ASPECTS OF DETERMINATION OF BENEFITS

The realistic determination of benefits also presents physical problems. It may be helpful to preface remarks concerning them by analyzing briefly some of the characteristics of alluvial rivers. Speaking in general terms, their banks have been built by the rivers themselves, and consist of a top layer of fines, usually in the silt classification, underlain by sands. The stream pursues a sinuous course of reversing bends connected by tangents of varying length. Deep water is found along the concave sides of the bends, which is where the major portion of bank caving takes place. The concave bank recedes by erosion, and the opposite "point" advances by accretion. The meanders also tend to move downstream, usually rather slowly.

Benefits Based on Study of Historical Bank Recession.—If maps of the river are available, showing the present bank lines and those of former years, comparison will indicate average annual rates of bank recession actually ex-

perienced. By classifying the lands destroyed, the average annual loss during the period studied of pasture land, cultivated areas, woodland and so on, can be determined. Consultation with the United States Department of Agriculture and with local agriculturists will permit fairly accurate estimates of net annual return from each type. Consultation with the owners will provide the data for estimating losses occasioned by destruction of structures, and the costs of the adjustments of levees, railroads, highways, bridges, pipe lines, power lines, and so on, made necessary by bank recession. Inasmuch as the stabilization project probably will have a life of 50 yr, it is desirable that the time interval between the bank lines compared be as long as the data permit. By reducing all experienced damages to a per annum basis, and assuming that the figure so derived will be applicable to the following 50 yr, a measure of the principal annual benefit attributable to stabilization of the banks results.

The method has certain shortcomings, however. Left entirely to its own devices, a river probably would cave its banks approximately as much in one half-century as it would in another. But the chances are that it has not been and will not be left entirely to its own devices. Instead, there may be in prospect a new or extended levee system, some artificial cut-offs, and perhaps a few flood control reservoirs on the stream itself or on its major tributaries. And it is almost certain that the development of its flood plain will have changed materially, during the years, so that even though the actual area of land annually destroyed were the same in the future as in the past, the resultant damages might be quite dissimilar.

Benefits Based on Projection of Future Bank Lines.—A better plan, where it is possible to use it, is to project future bank lines on the basis of past experience, and to evaluate losses of the various types enumerated previously on the basis of actual present-day (or, in special cases, even the foreseeable future) development of the specific areas which the study shows are likely to be destroyed if caving is allowed to proceed unchecked.

It is not likely that the positions of the bank lines of 50 yr hence can be estimated with satisfactory dependability unless the stream under consideration is unusually well-behaved, or unless its meanders are large. In the case of the ordinary stream, the continuing development and migration of meanders and occasional natural cut-offs lead to confusion when long-range prediction is attempted in a single step. It is necessary in such cases to proceed by fairly short steps. It is sometimes practical to project for 10 yr or 12 yr, and then to double the indicated movement, to develop a not-improbable 20 yr-to-24-yr future bank line. Even in developing the 10-yr-to-12-yr projection, it is best, where feasible, to study two previous river courses instead of only one; that is, to divide the study period into two parts, one representing where practical, a period of small floods and the other including a major flood.

As a check on the reasonableness of the projected bank line, it is always wise, after determining the area of land loss indicated for the length of river under consideration, to compare it with the area actually destroyed in a prior period of equal length. Unless there is some obvious reason to the contrary, they should agree, probably within 15%, if the projection has been carefully conducted. It is considered good practice to adjust the area lost as indicated by the projected bank line, to agreement with that actually destroyed in an identical historical period, and to adjust the total of indicated future damages

in the same ratio. This adjustment does not attempt actual modification of projected bank lines but it is purely mathematical.

ADJUSTMENT OF BENEFITS TO REFLECT CHANGES IN REGIMEN

On the general pattern previously outlined, certain other adjustments may be imposed if circumstances warrant. For example, by means of actual measurement of bank recession in a number of representative bends during the rise and fall of floods, a rough correlation between river stage and rate of bank recession can be derived. River stage is not the only factor affecting rate of bank recession, but it is probably the principal factor, and provides the only convenient criterion. If the period providing data for the projection of future bank lines happens to be one of exceptionally low high flow, stage duration data for the period used may be compared with those of the same period of equal length considered more nearly typical of normal regimen.

Used in conjunction with the relationship between rate of bank recession and river stage previously derived, the comparative stage duration data will permit adjustment of indicated rate of bank recession. Similarly, if flood control reservoirs of sufficient capacity to modify seriously the hydrographs of future floods are in prospect, the correlation between stage and rate of bank recession, in conjunction with comparative stage duration data for floods of record routed first through natural storage and then through reservoir storage, will provide a basis for adjustment of anticipated bank recession and of consequent damages.

ACCRETION

General Considerations.—After the direct damages that stabilization of banks would prevent has been determined, there remains the problem of discounting them appropriately to reflect the compensating effects of accretion. There are two schools of thought on the subject. One holds that in economic analysis, destruction of land by the river should be considered wholly uncompensated loss. This is not quite correct, but when it is realized that what usually is under consideration is the economic aspect of a project with a useful life of only 50 yr, the concept does not seem to be as far-fetched as it might at first appear.

At the other extreme, it is occasionally argued that even though the river width remains substantially constant throughout the years, there is no time lapse between erosion of bank and restitution of area by accretion. Whether it requires 10 yr or 200 yr for an accretion to form, the same processes that produce it today were operative 10 yr and 200 yr ago. At all times, there are maturing accretions to replace lost lands and restore lost revenues. This thesis is not quite correct either. River banks cave at moderate river stages, but they can accrete to elevations high enough to make them useful only when exceptionally high stages prevail, which is infrequently. Fresh accretion is also quite subject to destruction by caving. It may also be added to an island or be otherwise inaccessible. In all probability it will be ultimately usable for no higher purpose than pasture, and may be added to the lands of a farmer not engaged nor interested in cattle production.

Stabilization of the river's banks throughout a given reach does not normally preclude continued accretion. It prevents fresh losses of land already in use. Accretion will continue from caving banks in the reach itself during the years between inception and completion of the project (and their number may be large) from material derived from whatever deepening of channel follows the bank protection, from bank caving in reaches upstream from the project, and from erosion of uplands. It may slow down, but the character of the deposits may improve by including a larger percentage of silts. In addition, of course, any deepening of the stream would tend at least, to compensate (by lowering flow line) for any reduction experienced in elevation of deposits.

The coarser, underlying sands eroded by the river's current do not travel far from the point of their introduction into the stream, but the fertile, fine grained material which overlies them, travels great distances. Much of it may reach the sea, where a large part, perhaps most of it, is forever lost to man's use. In some cases it forms deltas. The Po River pushes its delta out into the Adriatic Sea at the rate of approximately 40 ft per yr, and permits the reclamation of approximately 300 acres of land per annum. The Shatt-al-Arab, formed by the junction of the Tigris and Euphrates, empties into a Persian Gulf whose present (1961) head is perhaps 170 miles seaward of its position of 5,000 yr ago. But even in such cases the deposit of fertile silt probably extends so far below depth of cultivation, that most of it has been lost to human use.

The Congo River empties into an ocean canyon so deep that no delta has been formed. The Rio Magdalena encounters great depths not far off-shore at Barranquilla. In 1935 and again in 1945, a disappearance of delta deposits into this abyss caused loss of a portion of the jetty system.

The enormous weight of sediment brought down by the Mississippi River through the ages, apparently has depressed the earth's crust at the river's mouth and may still be doing so. Deposits continue to increase the length of channel requiring dredging at the entrance to Southwest Pass, and some increase in land by reason of natural delta advance seems to be taking place. In general, however, the destruction of the high, fertile lands that border the alluvial streams of the United States appears to be destroying a valuable national resource. Part of it is destroyed permanently by being carried to the sea, and part of it temporarily by substituting sand bars for farm land.

Studies of Actual Cases.—In an effort to pass from the generic to the specific in the erosion-accretion cycle, several actual cases were examined in 1958-59. Selected at random on a fairly large stream for which aerial photographs of 1930 were available, and which had been photographed almost annually since 1945, their development was traced for some 24 yr to 26 yr.

At locality "A" in 1930, bank recession was just giving place to accretion. At locality "B," two sand bars had already appeared in a bend. At locality "C" a point bar was enlarging, and at locality "D," fill was depositing along the left descending bank.

In 1956, aerial photographs at "A" and "B" showed pasture (apparently unimproved). Backing up, year by year, it was found that the pastures apparently began around 1950. They definitely were not in existence in 1949. In the case of locality "B," the newly created pasture already had begun to cave into the river at its upstream end by 1956.

In the case of locality "C," the point bar developed rapidly between 1945 and 1950. By 1952 it had been almost entirely cleared, and the brush had been

neatly piled, apparently for burning. In 1954 the area appeared to be improved pasture. As of 1956 no threat of destruction by bank caving was evident.

In the case of locality "D" a wide accretion had formed by November 1945, and by the following October, there were willows and one small patch of what appeared to be grass, adjacent to the high 1930 bank-line. By 1948, grass was beginning to appear farther riverward, scatteringly in the upstream portion of the bar, in which the latter was beginning to cave. The downstream portion only showed willows. By October 1952, virtually the entire of the upper, grassed portion of the bar had been destroyed by bank recession, and only the wooded, downstream portion survived. The latter was still enlarging downstream as a bare sand bar. By September 1954 the surviving land between the 1930-45 shore lines (approximately half of the corresponding 1945 area) was perhaps 70% good woodland pasture.

Whether the four cases studied are collectively representative, they are of course problematical, but inasmuch as they were selected at random, they well may be. Considering their individual case histories, and the fact that a river is likely to do its most active bank caving within the limits of its contemporary meander belt, the conclusion seemed tenable that for the stream in question, a period of 30 yr may elapse between the time of bank caving and the use of the compensating accretion for bar pasture. The latter utilization, at least for the stream studied, appears to be the highest that may be anticipated. In the case of the caving of low woodland areas (areas too low even for pasture), it was felt that a lapse of only 10 yr need be assumed for restoration to low woodland; that is, 4 yr for accretion to the lower elevation considered necessary and 6 yr for growth of willows to marketable size.

Method of Discounting Benefits.—Just as the prevention by bank stabilization of the destruction of land by bank recession represents an arithmetic progression of benefits, so the restoration of revenues by natural accretion after a lapse of years discounts the computed benefits in arithmetical progression. The progression begins, in the case of the stream previously studied, approximately 10 yr after destruction by bank caving, by which time marketable willows may be assumed available. The net revenue thus yielded is small, but gives place in time to that derivable from pasture.

Pattern of Accretion.—It is not to be inferred from the foregoing that each specific area of land lost reappears eventually in its former location, first as low woodland, and later as pasture. There is no convenient means of determining what the nature of future accretion will be at a given locality. A possible compromise would be to assume that within the period represented by the life span assigned to the proposed project, land lost by caving will eventually reappear as a sand bar, woodland, unimproved bar pasture, and improved bar pasture. The reappearance of such land will be, substantially, in the same respective proportions currently exhibited by the areas involved in these categories with respect to the total area embraced by the four categories. This implies that cropped land destroyed will not reappear as such, but that portions of it will reappear in each of the four lower classifications named.

In the particular case of the stream for which the time lapses named previously were deduced, it was considered proper for various reasons to assume that if the unimproved pasture acreage was lost, only half would eventually reappear as unimproved pasture and that half would remain woodland. The assumptions both as to time lapse between loss and resumption of

revenue and the probable pattern of distribution of accreted lands between classes, will vary with the characteristics of the stream under consideration.

PRACTICAL CONSIDERATIONS AFFECTING STABILIZATION PLANNING

Where a large levee, a highway, or other expensive improvement has been endangered by bank caving, riparian interests frequently have revetted the bank or have built groins or training walls, and thereby have protected the areas immediately threatened. This often has contributed little toward the permanent stabilization of the river's banks because the installation of isolated and unrelated stabilization works ignores an important aspect of regimen previously noted herein, namely, the tendency of river bends to migrate downstream. As the result of this tendency, a caving bank in the course of time is blanketed by a sand bar, and the point of current attack moves down river. If the erstwhile caving bend was revetted, the installation is no longer useful because it is no longer under attack, and the total mileage of the caving bank is the same as it was before the revetment was placed.

This has been a hard lesson to learn. There are miles of expensive bank revetment behind wide sand bars along the major alluvial rivers. It requires firm resolve to expend large sums to protect a river bank where there is no imminent threat to an expensive improvement. But the testimony of mile on mile of inactive revetment irrefutably points to the necessity of a comprehensive plan of control if the problem is ever to be finally solved.

A comprehensive plan is the stabilization in such sequence as conditions may dictate, of every actively caving bend lying between points at which the positions of the banks of the stream are permanently fixed. Such fixed or control points may be at localities where the channel follows the bluff line or where for some other reason the bank is composed of non-erodible material. It may on the other hand be a town or a bridge, where, by reason of the values involved, present local alinement is established and will unquestionably be preserved at whatever cost, by artificial means. Between such control points, the order in which the intervening bends are stabilized, and the time of treatment of individual bends will be dictated by common sense and local conditions.

GENERAL PROCEDURE SUGGESTED FOR ECONOMIC ANALYSIS

In practically every instance, an alluvial river with a flood plain wide enough to make the cost of bank stabilization worth considering, is leveed. The procedure suggested for determining the economic soundness of stabilizing its banks is briefly as follows:

1. Using the most recent maps available, determine for the reach under consideration (which must normally extend from a "fixed" point to another "fixed" point) the extent and cost of the revetment, and pile dikes, required to protect all of the actively caving bank and the annual charges appertaining thereto. Where material savings in length of protection can be affected by means of artificial cut-offs across necks of land, they should be included in the plan. In general, as a rule-of-thumb for planning, cut-offs probably should be restricted to situations in which the bendway distance between opposite ends of cut-off is not less than three times the length of cut-off.

2. Planimeter and classify, as cropland, pasture, woodland, sand bar, and so on, all of the land in the reach considered, lying between the opposite levee lines (or between levee and high land where one bank does not require levees); that is, all of the land in the floodway.

3. From study of past maps, aerial photographs, and any other pertinent data available, project probably bank lines as far into the future as practical.

4. Planimeter by classes the lands indicated for destruction. Compute the loss of revenue corresponding to the indicated loss of land, and determine the cost of the levee set-backs and adjustments of power lines, pipe lines, bridges, railways, and highways required, the cost of moving other types of structures, and the value of non-removable structures that the recession will destroy.

In estimating cost of adjustments to levees, railways, roads, and such, follow the policy adopted by the owners. A levee probably would not be set back far enough to provide a 50-yr life. Several set-backs might be required. Similarly, pipe-line adjustments, road, and railway relocations may entail several moves during a 50-yr period. Include such additional structures, bridges, pipe lines, and so on, as the normal development of the effected area during the assumed life span of the proposed project appears likely to involve.

5. Discount revenue losses in (4) by estimated net revenues derived from accretions during the life of the project.

6. Planimeter actual area destroyed by bank recession within the reach under considerable during a prior period of years as nearly equal to the period used in the projection study as available data permit. Adjust computed losses as suggested by the ratio between experienced and estimated areas annually lost.

7. Estimate collateral benefits deriving from stabilization of banks. These may be quite varied in nature, and may include intangibles as well as those which readily can be evaluated. Removal of the threat of destruction by bank recession should improve the marketability and price of lands far from the river; should warrant the expense of improvements to riparian properties which would increase their productiveness; should improve the type of use to which lands under threat of loss are put; should eliminate the hazard of levee crevasses from rapid bank recession during floods; and should greatly reduce the maintenance costs of any isolated stretches of revetment existent in the reach. If the program involves cut-offs, the lowering of flood flow-lines may produce flood-control benefits of considerable magnitude by reducing back water overflow at mouths of tributaries. The horse-shoe lakes formed by the abandoned bend-ways may yield material recreational and wild-life benefits.

8. Determine the ratio between tangible benefits, (direct and collateral) and costs, bolstering it if and as necessary, by such eloquence with regard to intangibles as conscience will permit.

CONCLUSIONS

The destruction of good farm land by bank caving represents a loss of a natural resource which increases in importance with density of population, despite present (1961) agricultural surpluses. Its prevention, to be permanently successful, should embrace, in proper sequence, caving banks between points at which the bank position is already permanently fixed. Bank protection is so expensive that it can usually be attempted on a large scale, only on rivers

where wide and fertile or highly developed valleys are seriously menaced by bank recession. In the United States, rather comprehensive physical data usually have been accumulated with respect to such rivers. A fairly rational approach can be made from these data to the problem of determining where and how far the river will cave its banks in the next few decades if its banks weren't stabilized. The resultant annual losses in lands, revenues, and structures, and necessary relocations would be determined.

Estimates can be adjusted to give effect to the influence of engineering works, existing or anticipated, which will influence river stage and rate of bank caving.

The ameliorating influence of accretion in restoring revenue can be estimated on a fairly rational basis if a series of aerial photographs during a period of years is available.

Both revenue lost from bank caving and revenue restored by accretion becomes an arithmetical progression. Orthodox procedure in economic analysis converts present worth of the increasing annuities thereby represented, into average annual "equivalent" benefits, which later decreases as the interest earned by the annual benefits increases. A preferable basis of comparison would appear to be total benefits versus total costs, charging compound interest at the respective standard rates in each case.

Journal of the
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Proceedings of the American Society of Civil Engineers

CATHODIC PROTECTION OF MARINE TERMINAL FACILITIES

By Joseph Milano[1]

SYNOPSIS

The use of cathodic protection to inhibit corrosion of steel pilings at Port of New York Authority Marine Terminal Facilities is outlined. The integration and development of cathodic protection system design with various types of marine structures is described, together with a presentation of maintenance procedures and system economics.

INTRODUCTION

Today (1961), the Port of New York Authority's marine terminal facilities provide some 20% of the deep-water channel cargo berths in the Port of New York. About one-half of these wharfs and piers use steel pilings of varying designs.

The protection of these pilings to obtain reasonable longevity is a primary consideration in the initial selection of pile material. The choice of protection to apply is ultimately based on economics. In general, three types of protection are used, namely, paint coatings, concrete encasement, and cathodic protection. Herein will be described the integration of cathodic protection requirements with the overall design of marine structures.

From the first installation of a cathodic protection system at Port Newark (Newark, N.J.) Building 152 Wharf in 1953 to May 1961, the application, design,

Note.—Discussion open until October 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 2, May, 1961.

[1] Designer, Marine Terminals Planning and Constr. Div., Port of N. Y. Authority, New York.

and maintenance of these systems have changed considerably. Part of this evolution has resulted from changes in steel piling design concepts and part stems from experience gained from earlier designs.

CATHODIC PROTECTION - GENERAL

Cathodic protection, by definition, is the use of an applied current to prevent or retard corrosion of a metal surface. Corrosion of metals occurs as a result of direct chemical or electrochemical reaction with their environments. It occurs because in many environments most metals are not inherently stable and tend to revert to some more stable combinations of which the metallic ores, as found in nautre, are familiar examples.

The best known galvanic or electrochemical corrosion occurs when two dissimilar metals are in contact in the presence of a conducting medium. This is analogous to what takes place in the common wet-cell battery.

Each metal has its own level of activity called electrical potential. When two metals, such as steel and zinc, are connected and immersed in an electrolyte, such as sea water, electrons will flow from the zinc (the anode) to the steel (the cathode) because of this potential difference. The flow of electrons results in the release of metal ions from the anode surface into the electrolyte.

At the cathode, the electrons attract hydrogen ions from the sea water. These ions are neutralized by the electrons and a thin layer of nascent hydrogen is "plated-out" on the cathodic surface. The steel cathode is protected by the current flowing to it and by the insulating properties of the hydrogen film on its surface.

Because a piece of steel will corrode when immersed in sea water, there must be anodic and cathodic sites on the steel itself. The local corrosion cell may be a galvanic couple, oxygen concentration cell, or a cell made up of an anode at a stressed portion of the structural metal and a cathode at an unstressed one. As corrosion proceeds, the anodes and cathodes will shift and the corrosion will appear to progress as an even, over-all attack.

In an electrical series system, a break or dielectric introduced at any point in the circuit stops current flow throughout the system. Paint films and concrete encasements protect steel in this manner. The hydrogen film formed on cathodic surfaces is protective, but it usually is removed by combining with oxygen in the surrounding environment.

Another approach to the solution of this problem is the suppression of the corrosion currents. The anodic and cathodic reactions are usually interdependent and neither can occur without a flow of current. If a countercurrent from an external anode source is introduced into the corrosion circuit in sufficient amount to suppress the local currents, corrosion will be minimized. "Cathodic protection" thus depends upon neutralization of the corroding current and polarization and formation of layers of hydroxide and insoluble reaction products on the new cathodic areas.

SYSTEM DESIGN

As generally installed, the protective systems consist of five elements: (1) The anodes or source of protective current; (2) the cathodes or the steel surfaces receiving the benefit of this current; (3) the electrolyte, which in this

case is the sea water and silt through which the protective current flows; (4) the rectifiers, which are the source of the impressed protective potential; and (5) the circuiting required to establish optimum relationships among the other four elements.

The use of these five components and the analyses behind their selection and application will be related to the specific types of structures protected. Table 1 lists the structures as three general types: (1) Steel H-beam piling, (2) steel pipe piling, and (3) steel sheet-pile bulkheads.

Steel H-Beam Piling.—The wharf at Buildings 152, 153, and 154, Port Newark, was the first Port Authority construction to be supported on cathodically

TABLE 1.—MARINE TERMINAL FACILITIES WITH CATHODIC PROTECTION INSTALLATIONS

Facility (1)	Item Protected (2)	Installed Rectifier Capacity (3)	Anode Types (4)	Maintenance (5)
Brooklyn-Pier 1	Sheet Piling	20 KVA	Duriron Durichlor	Semi-annual inspection at selected points
Brooklyn-Pier 2	Pipe Piling	4KVA	Duriron Durichlor	Semi-annual inspection at each Anode Station
Brooklyn-Pier 10	Sheet Piling	3 KVA	Duriron Durichlor	Semi-annual inspection at selected points
Brooklyn-Pier 11	Sheet Piling	32 KVA	Duriron Durichlor	Inspected and tested by consultant firm
Brooklyn-Clinton Wharf	Sheet Piling	1.5 KVA	Durichlor Duriron	Inspected and tested by consultant firm
Port Newark-Wharf at Bldgs. 152, 153 and 154	H-Piling	21 KVA	Graphite	Semi-annual inspection at selected points
Hoboken-Pier A	H-Piling	33 KVA	Graphite	Semi-annual inspection at selected points
Hoboken-Pier C	H-Piling	30 KVA	Duriron Graphite	Semi-annual inspection at selected points
Erie Basin - Water Tower	Tank Interior	2 KVA	Magnesium	Annual inspection and maintenance by consultant firm

protected H-piles. The wharf consists of a reinforced concrete relieving platform supported by 232 bents, or parallel rows of piles each consisting of four vertical piles and one diagonally driven batter pile.

The piles are coated with bitumastic paint and the coating extends from the top of the piles down to the dredged bottom (mudline). All five piles in each bent are bonded electrically by means of welded reinforcing bars and a ground collector wire is welded to each bar.

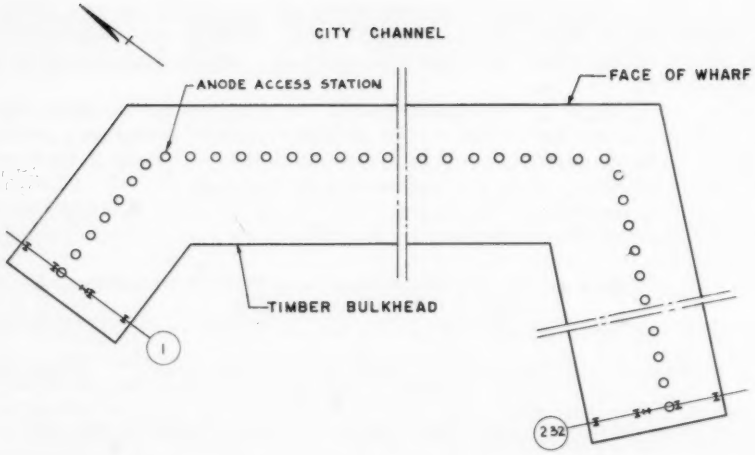

FIG. 1.—CONFIGURATION OF THE ANODE ACCESS STATIONS ON THE WHARF OF BUILDINGS NOS. 152, 153, AND 154, PORT NEWARK

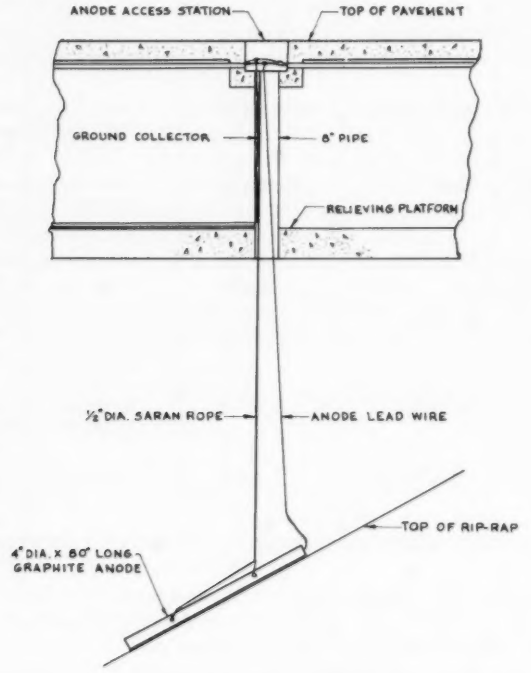

FIG. 2.—TYPICAL ANODE INSTALLATION UNDER THE WHARF OF BUILDINGS NOS. 152, 153, AND 154, PORT NEWARK

A total of 231 anode access stations are located on the wharf (Fig. 1) with an anode energized and supported from each station (Fig. 2). The anodes rest on a layer of quarry stone rip-rap. The station locations were selected to coincide as closely as possible with the centroid of the bents and thus minimize the possibility of a wide protective current range in the system.

The power requirements of the protective system were determined on the basis of 5 milliamperes per sq ft of steel surface in the water zone and 1 ma per sq ft of steel surface in the mud zone. The entire system is divided into seven areas with seven rectifiers operating in parallel. Each rectifier is of the air-cooled, selenium stack type, rated 200 amp and 10 v, except for one unit which is rated at 200 amp and 20 v. This latter rectifier is so located that the

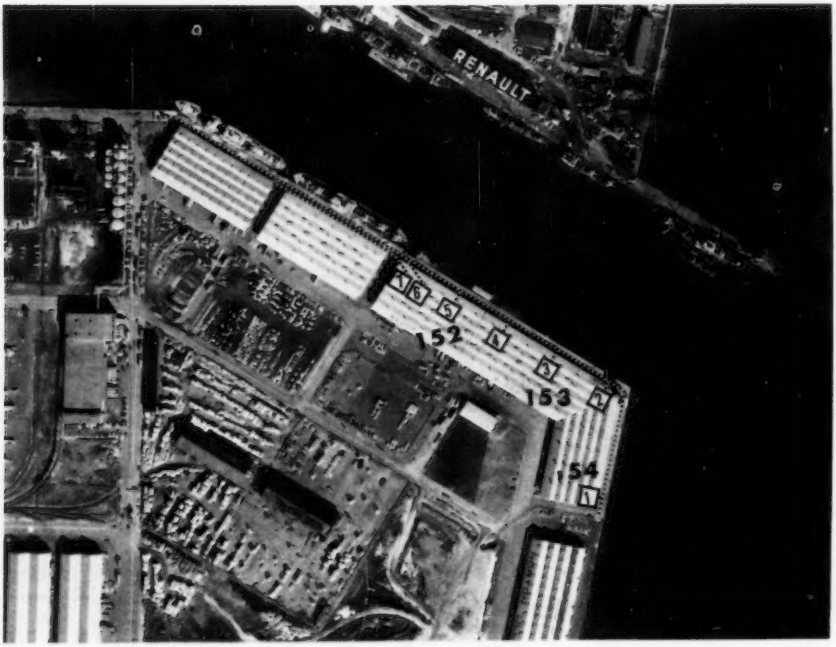

FIG. 3.—DISTRIBUTION OF RECTIFIER STATIONS

voltage drop to load requires a considerably higher voltage output than the others. It can be noted that the feeder length to Rectifier 7 predicates a higher voltage rating for this unit than for the others (Fig. 3).

Because of expected environmental changes in the water and irregular densities of mineral and organic coatings on the steel, it was necessary to provide a means of varying rectifier output. By providing 18 voltage taps on the secondary of the rectifier transformers, the rectifier output voltage can be adjusted from zero to its rated value (Fig. 4).

The rectifiers supply the anodes through a common pair of feeders (positive) and ground collector (negative) cables. At each anode access station,

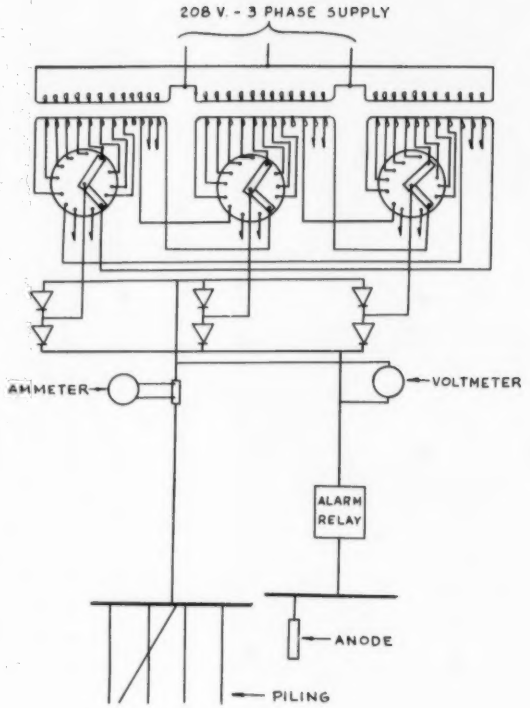

FIG. 4.—GENERAL WIRING DIAGRAM OF A THREE PHASE RECTIFIER

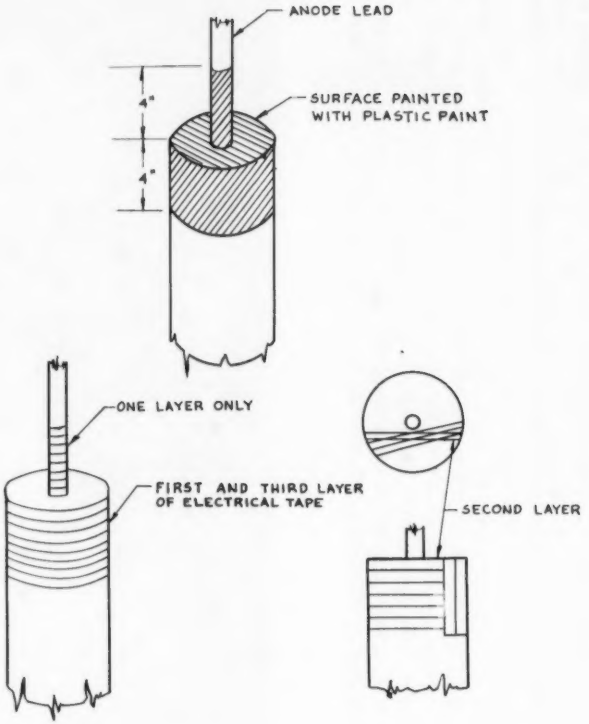

FIG. 5.—PAINTING AND TAPING DETAILS OF GRAPHITE ANODE

the anode lead wire is spliced to the feeder cable and the ground collector wire is spliced to the collector cable. A third wire, welded to the reinforcing bar, is terminated at each station and serves as a test lead.

The graphite anodes selected are 4 in. in diameter and 80 in. long. With a load of approximately 6 amp, the service life of these anodes is expected to exceed 10 yr. Neoprene jacketing was used on the rubber insulated lead wire and the insulation detailed at the point of anode connection (Fig. 5) to avoid premature failure due to the loss of anode material. Each anode is supported by a $\frac{1}{2}$-in. saran line and thus all underwater components of the protective system have chemical inertness as a common characteristic.

In order to provide supervision of the system, a circuit was designed to activate audible and visual alarms in case of fault or failure in the system. A shunt relay in each rectifier output is de-energized whenever the current falls below 50 amp. At a central alarm relay cabinet, there are seven tell-tale lights and an alarm bell rings until manually shut off.

The protective systems for Port Authority Piers A and C at Hoboken, N.J., essentially reflect the same design criteria applied to the Waterman Terminal with two notable exceptions. In the case of the two piers, it proved economical to use the superstructure as a part of the ground collector circuits since the building columns were co-linear with the steel pilings. The second distinct feature occurred in the design of Pier C. "Duriron" anodes, a high silicon cast iron with an integral lead assembly, were specified. The installed cost of the graphite anodes used in the two previous installations was higher than expected and obtaining a watertight lead connection proved difficult.

Steel-Pipe Piling.—An unusual design consideration in the construction of Brooklyn-Port Authority Pier 2 determined the use of three longitudinal rows of steel cylinder piling. The pier substructure basically consists of a reinforced concrete platform supported on concrete-capped wooden piles. However, a portion of the 625-ft by 350-ft pier is directly above the twin tunnels of the IRT Subway. To span the tunnels three rows of high capacity steel pipe piles were specified.

The protected piles are 12-3/4 in. OD steel cylinders filled with concrete and coated with a bitumastic compound. Electrical bonding was obtained by a grid of 3/4-in. plain round reinforcing bars, welded to each pile and lap welded to each other (Fig. 6).

The power capacity of the system was established on the same basis as on all other Port Authority protected facilities, that is, 5 ma per sq ft of steel surface in the water zone and 1 ma per sq ft in the mud zone. By 1957, the year Pier 2 was designed, this method of determining protective power requirements has been well established.

Sixteen anode junction boxes, each providing connections to two anodes, have been cast into the concrete deck. The anode boxes are equally spaced on 40-ft centers above the center row of steel piles (Fig. 6). A single 0-20 v, 200 amp selenium rectifier supplies energy for the entire system. Anticipating a wide variation in polarization voltage in the protected system, it was decided to provide twenty seven voltage selections at the rectifier output.

Complete protection of steel in sea water is attained when it has been polarized to a potential of minus 0.80 v or more electronegative referred to a silver-silver chloride electrode. Without a fine adjustment in rectifier output, it would be difficult to obtain a mean pile polarization potential of minus 0.80 v. In addition to the feeder voltage drop element, the area and rate of

sea water exposure per pile is not constant in this particular system, and thus a considerable number of variables existed which were expected to affect rectifier output.

In designing this protective system, the electrical interrelationship of the subways, tubes, and the steel piles was an important consideration. Accordingly, a study was undertaken of the magnitude and distribution of stray currents originating from the tunnels and recommendations were made in the pile protective system design that would eliminate the possibility of accelerating tunnel shell corrosion. It was determined that consideration of stray currents from the subway could be neglected and that currents from the cathodic protection system could be adequately contained by a thorough bonding of the steel and installing the anodes as close to the piling as possible.

In order to provide adequate current distribution and to satisfy the conditions cited previously, it was decided to suspend the anodes horizontally, 8 ft below mean low water (Fig. 7). At the inshore end of the pier, the mud line

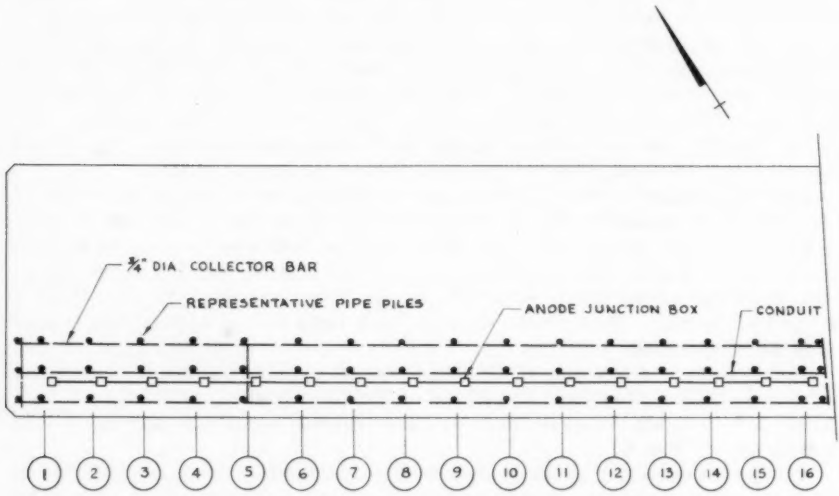

FIG. 6.—SCHEMATIC PLAN OF CATHODIC PROTECTION PILE BONDING AND ANODE JUNCTION BOX LOCATIONS FOR BROOKLYN PORT AUTHORITY PIER 2

rises above mean low water. This necessitated selection of two types of anodes; Duriron for the anodes in water and Durichlor for the anodes buried in silt. Duriron is attacked by anodically produced chlorine gas which cannot freely escape. Durichlor is a molybdenum bearing silicon iron alloy that was specifically developed for use in chlorine and chloride environments.

Failure of the rectifier output or supply voltage immediately energizes a horn alarm. This device is adjustable to operate in a range of 25% to 75% of rectifier rating.

Steel Sheet-Pile Bulkheads:—Brooklyn-Port Authority Pier 11, completed in 1958, was the first Port of New York wharf with steel sheet pile bulkheading

to be equipped with a cathodic protection system. The 2,100 ft wharf is retained by a Z-type piling bulkhead with a tieback system. Tierods, spaced every 9 ft, connect the bulkhead to the anchor steel sheet piling (Fig. 8).

Although the pile section joints are mechanically interlocked and under pressure, it was considered necessary to bond both the anchor piling and the bulkhead in order to minimize the potential drop between sections. A 1-in. diameter continuous collector bar was welded to each section. At both ends of the new wharf, approximately 430 ft of existing sheet piling was included in the corrosion mitigation system and consequently bonded to the new steel. The top of the bulkhead is capped with reinforced concrete and the rest of the steel has a bitumastic compound coating.

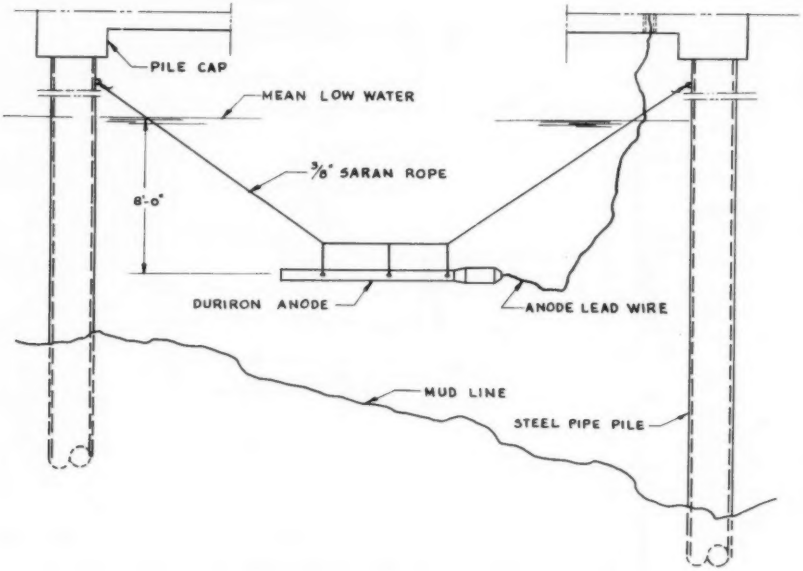

FIG. 7.—DETAIL OF ANODE SUSPENSION INSTALLATION UNDER BROOKLYN PORT AUTHORITY PIER 2 DECK

Complete cathodic protection is realized through a binary system. The primary seven rectifier circuit is designed to protect the water-side steel surfaces. A secondary three rectifier system protects the inboard steel areas. The selenium rectifiers are similar in design and rating to the Pier 2 rectifier, except for the primary system rectifiers which have a voltage output range of zero to fifteen volts. Each rectifier has a local audible alarm, eliminating the use of a central alarm panel and associated wiring.

There are 34 anode vaults located in the sand fill behind the bulkhead serving as anode connection points. Directly beneath each vault is an 8-in. diameter porous concrete inboard anode well (Fig. 9), and 34 Durichlor inboard anodes and 68 Duriron water-side anodes are spliced at the vaults. Each anode

is 3 in. in diameter by 60 in. in length. To minimize the number of anode stations, it was decided to design the anodes for maximum loads of 20 amp.

The inboard anodes are suspended vertically to a depth where the anode top is coincident with the mean low water mark, and a teflon jacket isolates the lead wires from chlorine attack.

A unique anode support structure was developed for use on the bulkhead water-side anodes (Fig. 10). The anodes could not be placed in the mud zone without creating operational problems during periodic dredging, and mounting

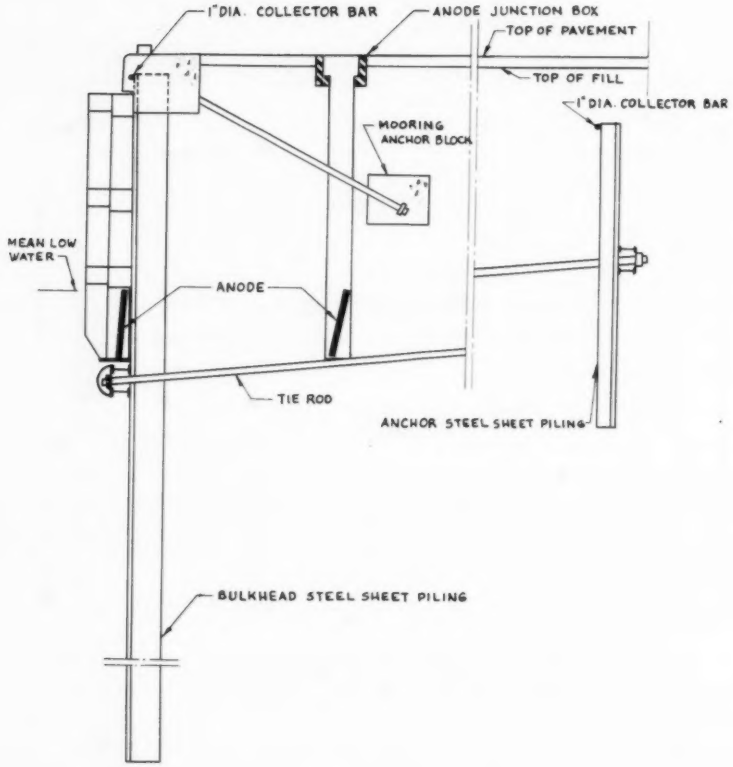

FIG. 8.—SECTION OF SHEET PILE BULKHEADING RETAINED WITH A TIEBACK SYSTEM, INDICATING THE LOCATIONS OF THE INBOARD AND OUTBOARD ANODES RELATIVE TO THE STEEL

the anodes outboard of the fendering system was mechanically impractical. Consequently, the anodes are supported on insulating shields, which are secured to the face of the sheet piling. This design, in effect, places the electrode a minimum of 2 ft from the bulkhead for current distribution and facilitates servicing. In computing the expected current distribution envelope of the

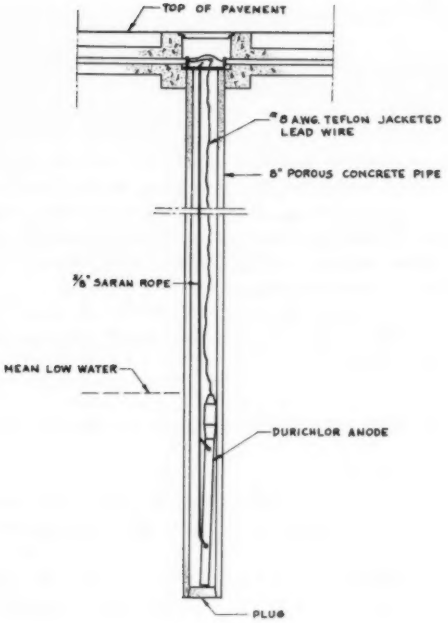

FIG. 9.—DETAIL OF INBOARD ANODE WELL AT BROOKLYN PORT AUTHORITY PIER 11

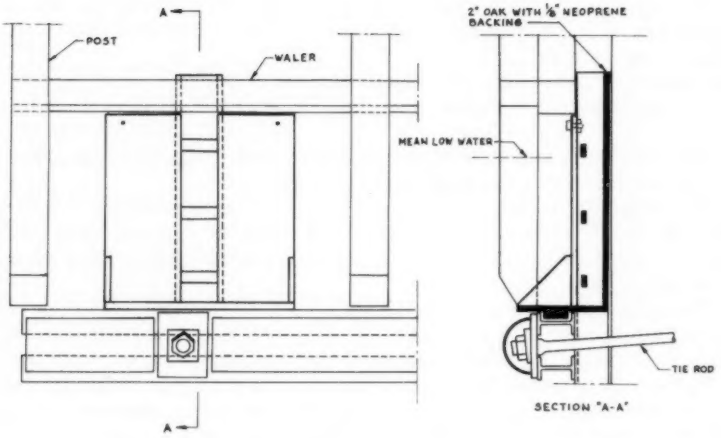

FIG. 10.—DETAIL OF OUTBOARD ANODE SUPPORT STRUCTURE AT BROOKLYN PORT AUTHORITY PIER 11

protective system, both the sand fill and the sea water were considered homogeneous mediums.

Relevant to system design, cognizance was taken of the effects on the steel of a ship berthed adjacent to a cathodically protected bulkhead. It was recognized that a portion of the current leaving the anodes could flow to a ship's hull and then back to the bulkhead. The ship would thus be anodic to the bulkhead and be theoretically subject to accelerated corrosion. Computations based on estimated resistance paths revealed that a typical hull current drain would be negligible and that no plating corrosion would occur unless a ship were tied up alongside a protected bulkhead for periods exceeding several months. Subsequent to construction of the wharf, rectifier tests indicated no change in current readings as a function of berth occupancy.

An analogous situation has been studied by the Royal Australian Navy,[2] where observations were made of the effect of berthing cathodically protected ships alongside unprotected ones.

SYSTEMS MAINTENANCE AND INSPECTION

The maintenance of a cathodic protection system involves considerably more than the sustenance of a fixed operational pattern. As was mentioned previously, some of the system components are variables and thus external control is required.

Upon completion of the cathodic protection installation, the rectifier voltages are individually adjusted to provide an optimum polarization potential at the protected steel.

The use of an integrated automatic control with constant monitoring at fixed points has been considered but has never been installed. Annual amortization of the initial cost of automatic control far outweights the value of reduced maintenance manhours.

Rectifier adjustment or system "balancing" is the first maintenance assignment on a system. In every case, with the exception of Brooklyn Pier No. 2, a satisfactory balance was achieved solely by "trial and error" variation in rectifier output voltage.

The balancing procedure has been essentially similar at all installations and can be described as follows:

1. The condition of a silver-silver chloride test electrode is checked for calibration against a standard calomel electrode.
2. The negative terminal of a high resistance potentiometer-voltmeter is connected to the test ground wire, where available at the test anode station. At installations without integrated test leads, a portable meter lead is clipped to the protected steel.
3. The meter positive terminal is connected to the test electrode with a waterproof conductor. At a point as close as possible to the steel element with the negative terminal connection, the electrode is lowered into the water.
4. Three vertical plane readings are taken at a point just below the water's surface, at the mudline, and at a point midway between the first two. This procedure is repeated at every selected test site. The test locations are either

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2 "Electrical Bonding of Cathodically Protected to Unprotected Ships," by G. A. Bennett, N. S. Dempster, and A. J. Wallace, Corrosion, Vol. 15, No. 11, November, 1959, p. 47.

predetermined during the design stages or are field selected to give a representative picture of the applied protective potential.

5. The rectifier outputs are adjusted (trial and error) so that the protective potential readings range between -0.80 and -0.92 v. Readings more positive than -0.80 indicate that complete protection is not being obtained, while

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ELECTRICAL TEST DATA

SHEET ___ OF ___

FACILITY Brooklyn Pier No. 2 DATE 8/25/ 1959

TYPE OF TEST Pile Potential & Current Readings

ELECTRICAL SERVICE

EQUIPMENT LOCATION

APPARATUS TESTED

(MANUFACTURER) (NAMEPLATE RATING) (STYLE) (SERIAL NO.)

South North

				1	2	3	4	5	6	REMARKS
Loc.	Bot.	Mid	Top	#1 Anode	#2 Anode	Bot.	Mid	Top		
3a										
2a	0.845	0.890	0.880							
1a	0.865	0.875	0.880							
1	0.804	0.806	0.900	8.6	8.5	0.804	0.900	0.905		
2	1.00	0.904	0.904	8.5	9.1	1.09	1.15	1.10		
3	1.14	1.08	1.01	8.7	8.7	1.15	1.12	1.10		
4	1.04	1.05	1.02	9.1	9.0	1.22	1.23	1.18		
5	1.29	1.29	1.16	9.0	8.2	1.46	1.46	1.44		
6	1.06	1.06	1.04	6.7	6.5	1.20	1.26	1.26		
7	1.06	1.06	1.04	6.5	6.4	1.24	1.26	1.24		
8	1.13	1.10	1.05	6.9	6.9	1.20	1.10	1.09		
9	1.04	0.980	0.960	7.0	5.8	1.20	1.20	1.18		
10	0.920	0.880	0.882	11.3	9.0	1.02	1.5	1.20		
11	0.930	0.941	0.935	7.6	7.9	0.904	0.907	0.906		
12	0.940	0.946	0.939	6.2	6.5	0.908	0.905	0.904		
13	0.820	0.840	0.840	6.0	6.0	0.900	0.901	0.901		
14	0.900	0.900	0.885	5.3	5.5	0.841	0.847	0.846		
15	0.902	0.920	0.900	4.5	4.4	0.900	0.900	0.900		
16	0.780	-	0.780	2.4	2.4	0.840	0.842	0.842		

ADDITIONAL INFORMATION:

Test Cell on black was 0 volts above standard

" " " white " 0 " " "

Rectifier 192 amps. 6.1 volts taps - top 4 bottom 2

TEMPERATURE ___ °F TESTER: ___

FIG. 11.—TYPICAL SET OF PERIODIC PILE POTENTIAL AND RECTIFIER READINGS

voltages more electonegative than -0.92 indicate that energy is being wasted, anode loss is accelerated and blistering of the bitumastic coating is a possibility. A set of pile potential and rectifier readings is shown on Fig. 11.

6. At Brooklyn Pier No. 2, with a single rectifier supplying all the anodes, the optimum "balance" that could be obtained by rectifier voltage output regulation resulted in a protective potential range of -0.71 to -1.10. With the addition of insulated nichrome wire resistance coils in the first nine outshore anode boxes, the potential range was compressed within the acceptable limits of -0.78 v to -0.94 v.

As soon as a new system is adjusted and is operating satisfactorily, it becomes a part of the facility electrical maintenance program. Each marine terminal facility records the voltage and current output of its rectifiers and makes a weekly report of the readings of the Central Maintenance Engineering Division. Here the data is charted for each rectifier and any deviations from the norm are immediately investigated by a trained staff. The seasonal changes affecting the conductivity of the electrolyte can be noted (Fig. 12). By using

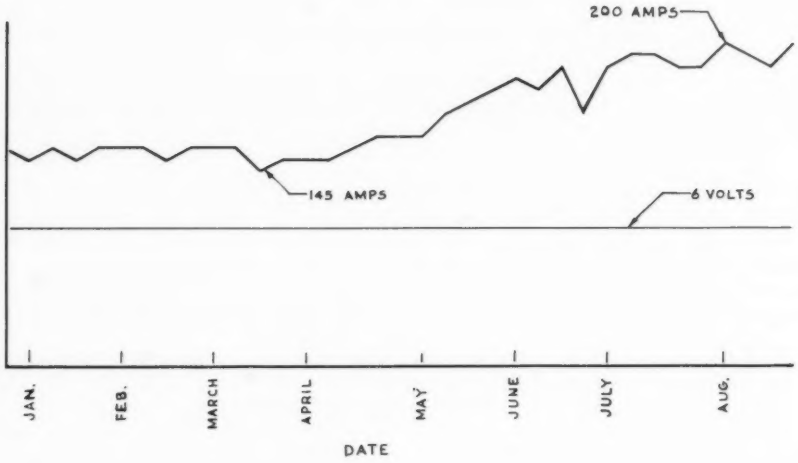

FIG. 12.—PORTION OF THE CONTINUAL CHARTED OUTPUT DATA FOR A SELECTED RECTIFIER

graphs it is often possible to differentiate between an internal system disturbance and fluctuating environmental conditions.

Environmental changes are normally caused by seasonal variations in water temperature, level and salinity, and also by heavy local rainfalls. Occasionally, a combination of low tide, cold weather, and a high proportion of fresh water will reduce rectifier output current to the level where the alarm circuit is tripped. An abrupt increase in the load on a rectifier usually indicates a physical fault within the system. There are also gradual changes due to attrition of anode surface area and a complementary accumulation of cathodic inhibitors which lower the corrosion rate by reducing the cathodic area. The reduction of the cathodic area is accomplished by the formation of insoluble compounds on the steel surfaces.[3]

[3] "Factors Influencing Corrosion," by G. V. Akimov, Corrosion, Vol. 15, No. 9, September, 1959, p. 23.

Semi-annually, personnel of the Central Maintenance Engineering Division inspect each Port Authority cathodic protection system. The alarms are checked and electrode-to-pile potential readings are taken at representative points. A separate annual physical inspection is made by an engineer from the Inspection and Safety Division who examines the piles, conduit, wiring, rope, and anodes on a sampling basis. Thus far there have been no anode replacements required due to appreciable decrease in diameter or length. The bitumastic coating on the pilings is examined for signs of blistering and scraping by ice and other debris.

Some difficulty has been experienced with the waterside anode support structures on the sheet pile bulkheads at Brooklyn Piers 1 and 11. Tidal action and debris, combined with installations which deviated from design, have

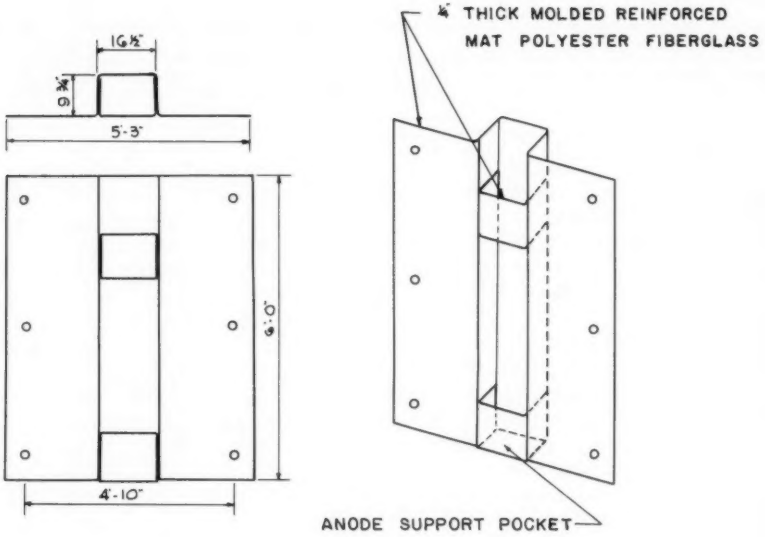

FIG. 13.—DETAIL OF A FIBERGLASS OUTBOARD ANODE SUPPORT STRUCTURE

resulted in the loss and destruction of some of the support structures. At Pier 11, which is located in a protected basin, the damaged anode supports and shields were replaced with 6-ft by 8-ft neoprene sheets. The rapid tidal water flow at Pier 1 precludes the use of neoprene sheets. Consequently, at the latter pier rigidly bolted fiberglass units (Fig. 13) will be substituted for the lost wooden units.

ECONOMICS

The most important criterion that determines the type of protection that will be used on steel marine piling is the economic one. The theory of cath-

odic protection would never be realized in an actual installation and would still be an academic curiosity if the expected advantages did not compensate for the costs.

The installation or construction costs will necessarily reflect the spatial configuration of the steel piling as well as the type of environmental exposure

TABLE 2.—INSTALLATION AND ENERGY COSTS

Structure Type (1)	Facility (2)	Installation Cost Per Square foot of Protected Surface (3)	Annual Energy Cost Per square foot of Protected Surface (4)
H-Beam Piling	Bldg. 152, 153 & 154 Wharf	$0.16	$0.0050
Pipe Piling	Pier 2	$0.21	0.0054
Sheet Pile Bulkhead	Pier 11	$0.13	0.0075

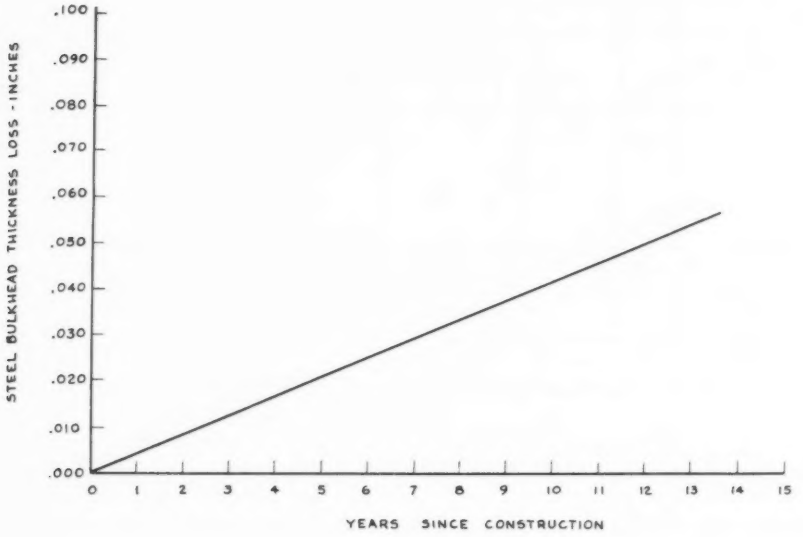

FIG. 14.—CORROSION OF SHEET STEEL BULKHEADING AT ERIE BASIN PORT AUTHORITY PIERS

on the steel surfaces. For the three types of structures previously described, the installation and energy costs are shown in Table 2. These costs do not include the application of bitumastic paint.

The electrical power costs shown are based on recorded power consumption over a number of years. Besides the expected yearly variations due to

seasonal fluctuations on water conditions, it was noted that power usage is highest during the initial years of installation. This is due to the lack of calcareous salt deposits on the new piling.

In 1958, the Port Authority acquired Beard's Erie Basin property along the north shore of Gowanus Bay in Brooklyn. This facility has five steel sheet pile bulkheads, totalling 2600 linear ft and installed during the 1946-1950 period. None of these bulkheads were cathodically protected and thus an opportunity was provided to study the corrosion characteristics of steel bulkheading. Core samples were taken of the piling at regular spacings and test pits were dug on the inboard side of the bulkheads to examine the conditions of the tie rods. Whereas the data collected are still (1961) under evaluation, study of the corrosion pattern reveals the average rate of corrosion experienced at each bulkhead to be approximately the same. When all steel samples are averaged and plotted against years since construction, the resulting relationship is as shown in Fig. 14. This type of study is correlated with estimated costs of cathodic protection system installation and operation, and data obtained from the Port Newark test piles.[4] Planning management is thus enabled to appraise the value of cathodic protection systems on an economic basis.

SUMMARY

The problem of protecting steel pilings at wharfs and piers to obtain maximum longevity compels serious considerations of impressed cathodic protection systems. Although the principle of corrosion mitigation by controlling the potential difference of the protected metal with respect to the surrounding medium is well known, several complications arise when this principle is applied to marine substructures.

The cathodic protection system must be carefully designed so that the geometric relationship of the anodes with respect to the cathodic steel will result in maximum anode loading and an optimum polarization potential. Also the constantly changing nature of the sea water or electrolyte requires provision for sensitive rectifier adjustment in the initial design. Complementing an efficient installation, a maintenance and inspection program is necessary to monitor deviations which affect system balance. Where there are other structures in the vicintiy, steps must be taken to insure that those structures are not damaged.

Any description of cathodic protection would be incomplete without consideration of the economics involved. The protective system costs of an installation should be evaluated in terms of the benefits to be gained.

It is not axiomatic that cathodic protection is desirable at every steel pile installation, but when properly designed and installed, it has accomplished its purpose effectively.

4 "Electric Cathodic Protection," by E. P. Pitman, Electrical Construction and Maintenance, Vol. 56, August, 1957, p. 94.

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HURRICANE WINDS FOR DESIGN ALONG THE NEW ENGLAND COAST

By C. S. Gilman[1] and V. A. Myers[2]

SYNOPSIS

Several hurricanes during the past 25 yr (1935-1960) have emphasized the exposure of the southern New England shore to damage from hurricanes. The meteorological cause of the high water and waves is found primarily in the wind field over the sea. Detailed estimates of the wind fields that existed in these and other hurricanes lead to some reasonable judgments as to the severity of hurricanes that might be expected in the future.

INTRODUCTION

The basic facts about hurricanes are generally well known. However, some recapitulation may be in order: Hurricanes are formed only over the ocean and only over very warm water. But, once started, they may travel great distances from their origin. There are many variations, but one of the typical patterns is for the storm to move steadily at a moderate rate of 10 mph or less for several days or even a week toward the west at a tropical latitude. In the western part of the ocean they swing in a great arc toward the northeast and speed up in their forward motion. This happens in both the Atlantic and Pacific Oceans. New England, with some protrusion of the coastline, intercepts some of these recurved hurricanes; Japan, with its more exposed posi-

Note.—Discussion open until October 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 2, May, 1961.

[1] Chf., WMO Hydrometeorological Special Fund Proj., Chile; formerly Chf., Hydrometeorological Sect., Weather Bur., U. S. Dept. of Commerce, Washington, D. C.

[2] Chf., Hydrometeorological Sect., Weather Bur., U. S. Dept. of Commerce, Washington, D. C.

tion, and with a higher frequency of hurricanes in the Pacific (called typhoons there) intercepts many more of them.

A census[3] of hurricanes over the past 75 yr (since 1885) shows an average annual frequency of 4.6 full-fledged hurricanes per yr in the Atlantic, including the Gulf of Mexico. (Almost all of these have occurred during the July through October season.) There is significant variation from year to year in the frequency but no important long-term trend. One of these reaches New England on the average of once every 5 or 6 yr; many of these have weakened to the point where, when viewed after the fact, they are not of much significance as damage-producers. When a hurricane is approaching, however, there can be no positive assurance that this weakening will take place. In fact, hurricane Carol of 1954 appears to have regenerated and intensified somewhat on approaching the New England coast.

Hurricanes have always been part of the New England climate. An extremely high tide at Providence, R. I., in 1635 almost surely was caused by a hurricane.

RECENT SEVERE HURRICANES

There have been five noteworthy hurricanes in southern New England in the past 25 yr (since 1935). These were in 1938, 1944, two in 1954, and Donna in 1960. Three of these produced large water-level rises in Narragansett Bay, R. I. The 1938 storm was the most disastrous. Edna of 1954 passed farther to the east and produced its greatest impact on Cape Cod, Mass. It was this storm that toppled the steeple of Old North Church in Boston, Mass.

An important part of estimating what force future severe hurricanes might attain is to first reconstruct the past storms. The reconstructed wind patterns are also required by the hydraulic specialists to check out and calibrate relationships between wind and tide for a particular geographical locale. A list of central pressures and other parameters for United States hurricanes obtained by reconstruction has previously appeared.[4]

In reconstructing hurricane wind patterns over the sea all along the United States coast, the primary method used by the writers and their colleagues has been the indirect one of extrapolating coastal observations to sea. Throughout the world, weather maps over ocean areas are prepared from weather reports radioed every 6 hr from cooperating merchant vessels. But it is perhaps a credit to the warning service that now (1961) not many reports are received from ships in the severe part of a hurricane; they manage to stay out of the way. During the 1944 hurricane there were no reports at all because of wartime radio blackouts, nor was it found possible to correct this by post-examination of ships' logs.

A brief summary, partly from previous work,[5] of some of the extrapolation techniques used to carry coastal observations to sea will be given. First a track of the particular storm center is determined on a large-scale map. The track adopted is the one that, on a geometric basis, will most nearly account for all the available pressure and wind observations. To do this it is assumed

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[3] "North Atlantic Tropical Cyclones," by G. W. Cry, W. H. Haggard, and H. S. White, 1959, Weather Bur. Tech. Paper No. 36.

[4] "Meteorological Aspects of Storm Surge Generation," by D. Lee Harris, Proceedings, ASCE, Vol. 84, No. HY 7, December, 1958.

[5] "Characteristics of United States Hurricanes Pertinent to Levee Design for Lake Okeechobee, Florida," by V. A. Myers, 1954, Hydrometeorological Report No. 32.

that the fields of pressure and wind are both approximately circular and move forward in a fairly regular, rather than erratic, manner. The weakening or intensification of the storm with time is also assumed to be fairly regular rather than erratic. As example of a geometric rule, the wind direction at an observing station will turn at a rate which is proportional to three factors: (a) How fast the storm is moving forward, (b) the distance from wind-observing station to the center of the storm, and (c) the sine of the angle between the direction of storm motion and the bearing of the observing station from the storm center. As might be imagined, one observation close to the track is more helpful than a dozen observations a long distance from the track. The paths of the four severe hurricanes reconstructed in the indicated manner are shown in Fig. 1.

The next step is to construct the field of sea-level pressure. The pressure fields are built up around the adopted positions of the storm center on the track from the available pressure observations. It is assumed that the radial profiles of pressure have a characteristic shape defined by an empirical formula, and that the pattern as a whole is relatively circular and changes and moves in a non-erratic manner. Fig. 2 shows a characteristic time-graph of the central pressure rise in a northward moving hurricane, the 1938 storm. Fig. 3 shows radial profiles of the pressure as reconstructed to the east of the same storm (the most severe quadrant). Fig. 4 shows selected corresponding plan views of the sea-level pressure.

The pressure values are needed as a tool in estimating the winds and are also applied directly to surge computations. From hydrostatic reasoning the water level rises about 1 ft for 1 in. (of mercury) depression of the sea-level pressure below normal atmospheric pressure.

Next the wind-speed field is developed. There have been basically three techniques for this. The first attempts to fit the available wind observations into circular patterns with characteristic radial profiles, as was done with the pressure observations. Before doing this the wind observations from coastal and other land stations are reduced to a common height of anemometer and the common frictional exposure of "over water" by empirical adjustment factors.

It would be well to point out that all the work on wind speed is done, insofar as possible, with averages over 10-min intervals. The maximum gust velocities, which are dramatized in newspaper reports, may be 40% or more higher. The gust velocities are of critical engineering significance for the design of towers and other such structures, but are of no known importance for surge generation.

The second indirect technique of deriving a wind field is a conversion of the pressure field to wind. The gradient wind, which is a theoretical frictionless and unaccelerated wind and a standard concept in meteorology, is computed from the pressure field. This is then reduced to surface winds by special empirical factors that apply to hurricanes.

The third method of estimating hurricane winds over the sea when observations are lacking is the construction of dynamic trajectories.[6] The universal equations of motion of hydrodynamics apply to the air in the surface layer of a hurricane. Where a wind velocity on the edge of a hurricane is available from a ship's report, subsequent velocities downstream and closer in toward

[6] "A Graphical Method for Computing Horizontal Trajectories in the Atmosphere," by H. V. Goodyear, Monthly Weather Review, Vol. 87, May, 1959, pp. 188-195.

FIG. 1.—RECONSTRUCTED TRACKS OF FOUR MAJOR HURRICANES REACHING THE NEW ENGLAND AREA

the storm center are computed by constructing an air trajectory based on the equations of motion. The factors which govern the motion are the initial velocity of the fluid, the pressure gradient, the coriolis force, and the frictional stress. Centrifugal force may also be important, depending on the choice of coordinate system. The big uncertainty in this procedure is the frictional stress. Friction coefficients derived from simultaneous wind and pressure observations in a hurricane over Lake Okeechobee, Fla., were used for this.

Fig. 5 shows selected wind patterns thus reconstructed for the 1938 hurricane, Fig. 6 for the 1944 hurricane, and Fig. 7 for Carol of 1954. The arrows on the figures show the probable wind directions. The winds in these and all

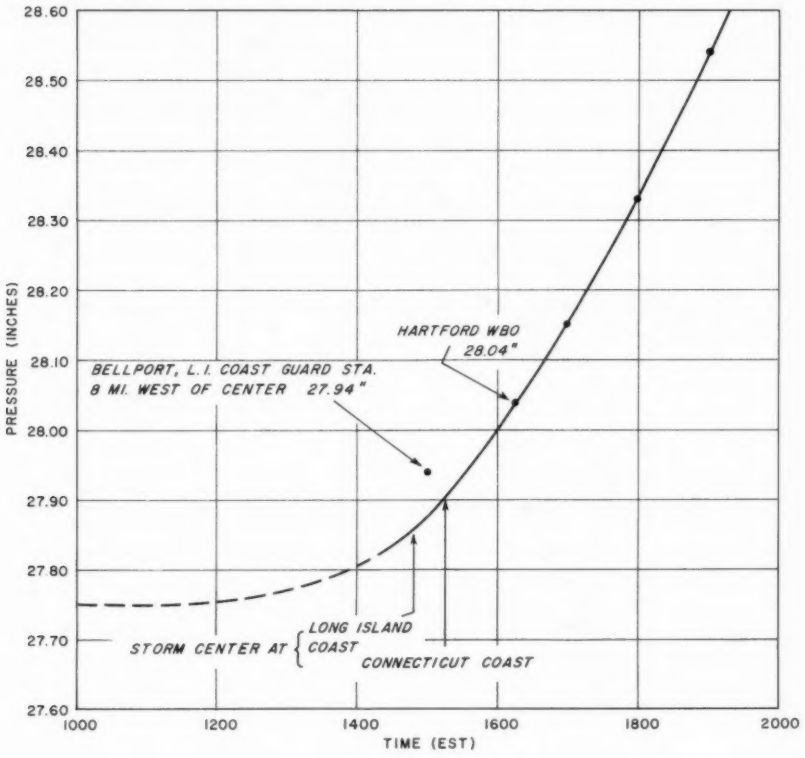

FIG. 2.—CENTRAL PRESSURE OF HURRICANE OF SEPT. 21, 1938

other figures are at 30 ft above water. A more complete series for the first two storms appears in a recent Weather Bureau, United States Department of Commerce (USWB) publication.[7]

Wind patterns such as the foregoing are used to compute surges on the open coast. But at the heads of bays there are additional factors. The surge

7 "Surface Winds Near the Center of Hurricanes (and other cyclones)," by H. Graham and G. N. Hudson, U. S. Weather Bur., Natl. Hurricane Research Proj. Report No. 39, 1960.

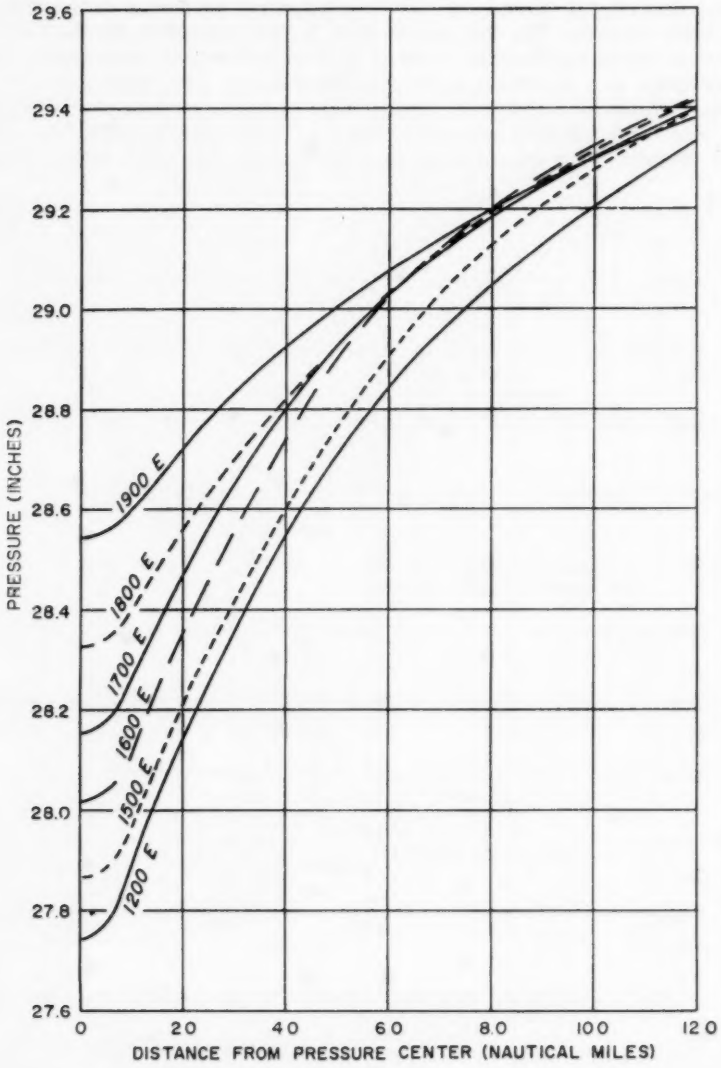

FIG. 3.—PRESSURE PROFILES TO EAST OF CENTER OF HURRICANE OF SEPT. 21, 1938. TIME (EST)

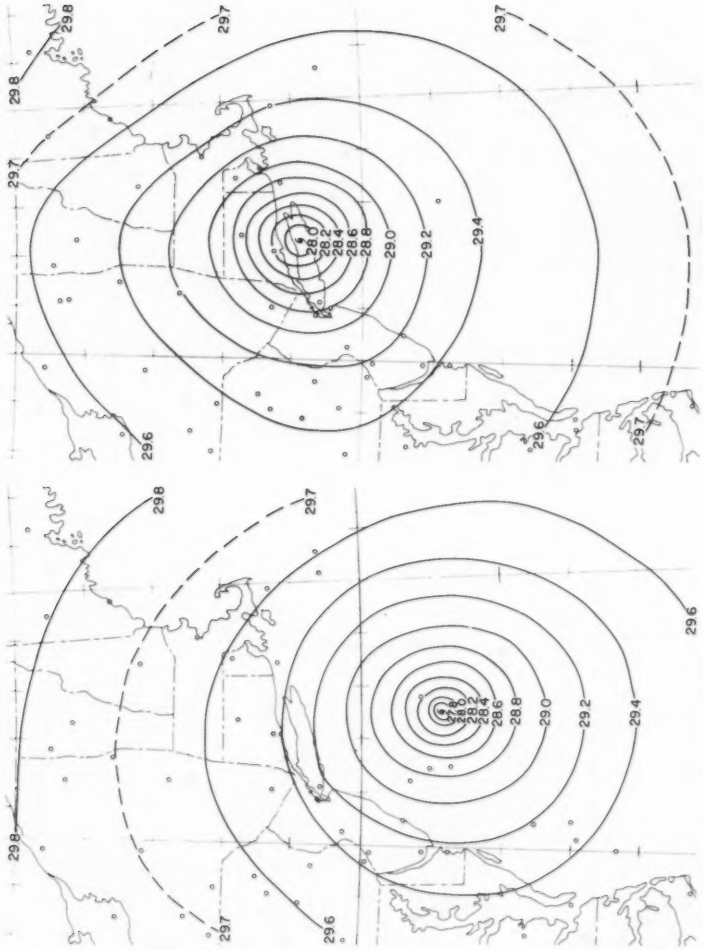

FIG. 4.—SEA-LEVEL PRESSURE (INCHES) HURRICANE OF SEPT. 21, 1938

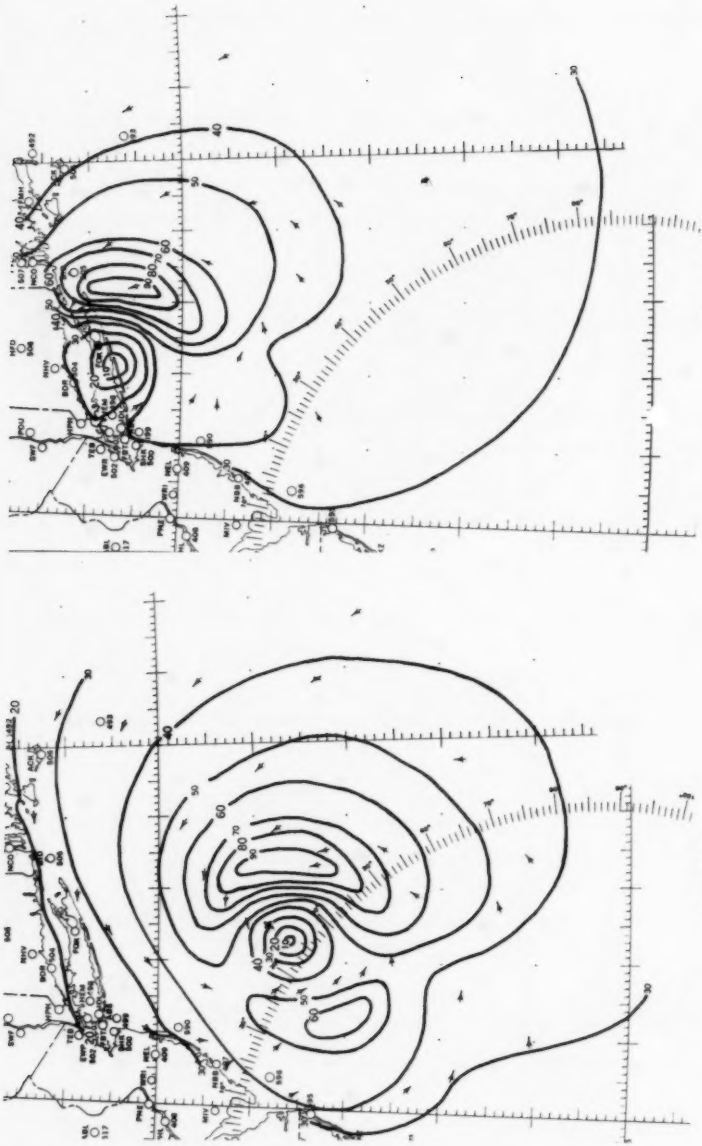

FIG. 5.—RECONSTRUCTED WIND FIELD (MPH) HURRICANE OF SEPT. 21, 1938

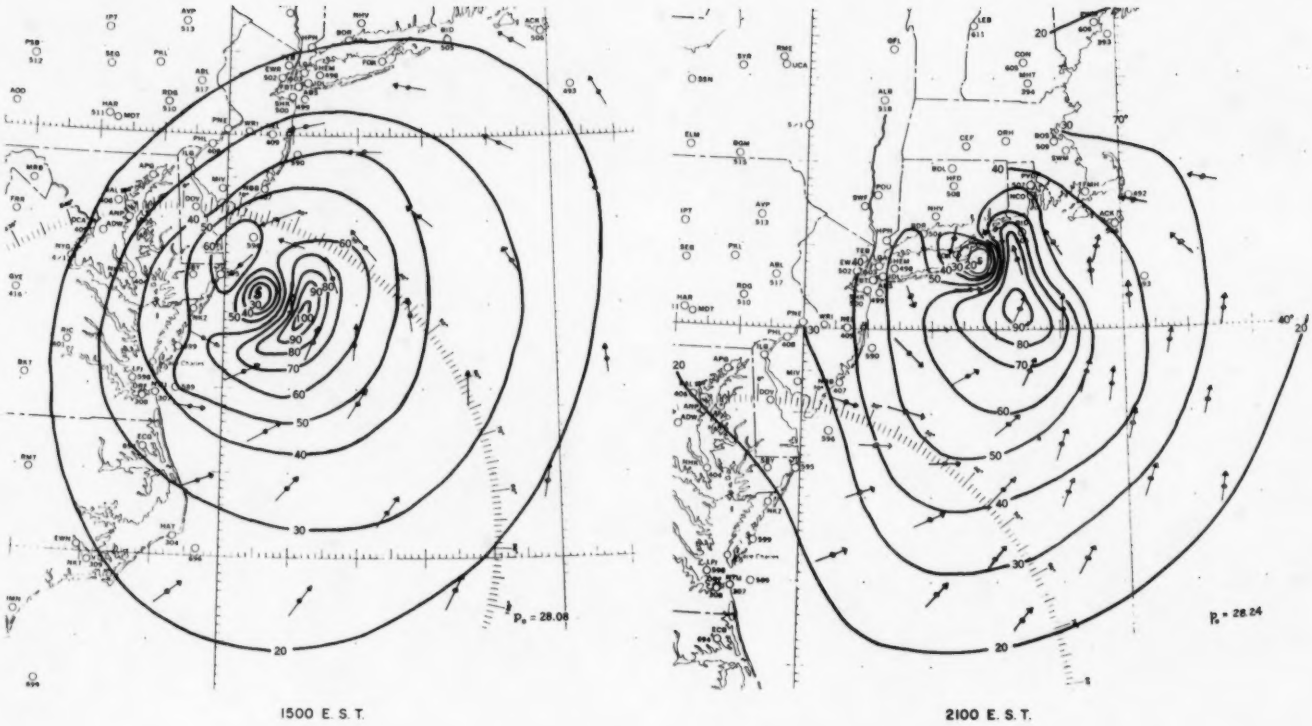

FIG. 6.—RECONSTRUCTED WIND FIELD (MPH) HURRICANE OF SEPT. 14, 1944

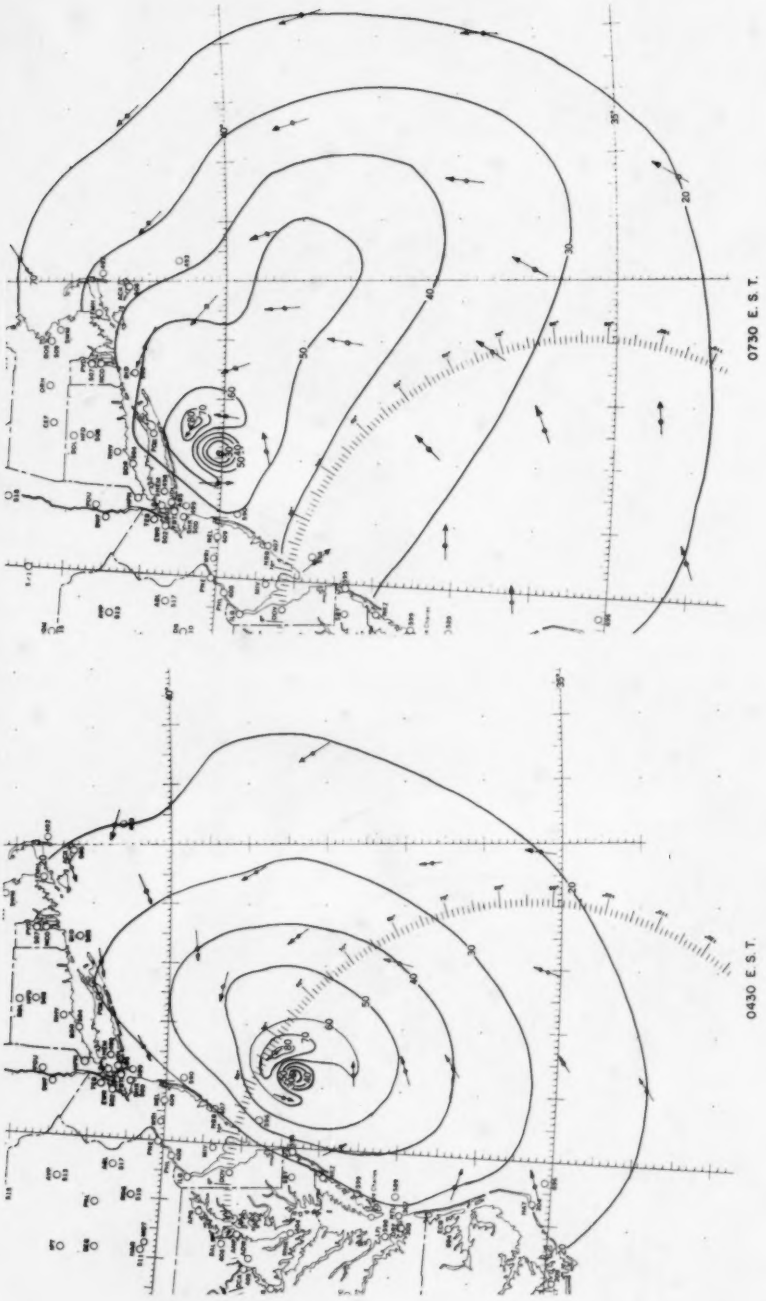

FIG. 7.—RECONSTRUCTED WIND FIELD (KNOTS) HURRICANE CAROL, AUGUST 31, 1954

at the mouth of Narragansett Bay, for example, proceeds up the bay as a gravitational wave. On top of this there is an additional setup of the water from the stress of the wind over the bay. To evaluate this additional stress, detailed patterns of winds over the bay were developed. An example of a detailed wind pattern for Narragansett Bay is shown in Fig. 8. These were synthesized by constructing the wind pattern that would have existed in this region had it been open water, then reducing by appropriate frictional factors. Such a procedure lacks scientific exactness; it is a case of doing what can be done with what is available, tempered by some judgment.

There is a new development in the observation of hurricanes over the sea—radar. Hurricane Donna of 1960 was kept under radar surveillance. The USWB, as part of its coastal radar network, has a powerful weather-search radar at Boston, on Nantucket Island, and at New York, N.Y., which can "look" 150 miles to 200 miles in all directions. During hurricane threats comparable radars belonging to the Air Defense Command are manned by radar weather specialists from the Weather Bureau and furnish reports to the Hurricane Warning Center. Such radar stations are located at some very strategic points on the coast.

Fig. 9 is an example of what the radar "sees" in a hurricane. The radar photograph of hurricane Donna of 1960 clearly shows the open eye of the storm. The white bands swirling into the center are rain and cloud areas. By means of continuously available pictures like this on the radar scopes, it is possible to keep close tabs on any hurricane that comes in over the Continental Shelf. The radars do not give the winds directly and their greatest value is in showing where the storm is located. However, one can learn something about the winds by following the positions of individual precipitation echo cells. These cells, of course, do not just drift with the wind, but are themselves formed, dissipated, and propagated. Further, their drift probably more nearly portrays the wind at some upper level than at the surface. Developmental work is being done on the relation of radar cell motions to wind by the USWB and at several universities.

MAXIMIZED HURRICANES

The judgment of two professions, meteorology and engineering, is involved in maximization for design purposes. Each is a profession requiring the full time of its practitioners and each has a method and language that are only incompletely understood by the other. At the same time there must be a large common area of understanding if an engineering project in which weather is a major factor is to accomplish its purpose. This is especially true when the project involves the conflict between the safety of large numbers of people versus the spending of large amounts of public funds. One way at arriving at this common understanding is through an interchange of ideas and problems.

In maximizing weather events for design purposes, be it a hurricane breakwater, a tower, a dam, or a highway culver, it might be said that one seeks to distinguish the possible from the impossible, or the probable from the improbable, or the likely from the unlikely, or, coming on down the scale, simply the frequent from the infrequent. The character of the project determines which of these distinctions is to be made. The casual student of weather processes could start the job by setting outer limits. One weather event would be clearly "possible" because something close to it had recently occurred.

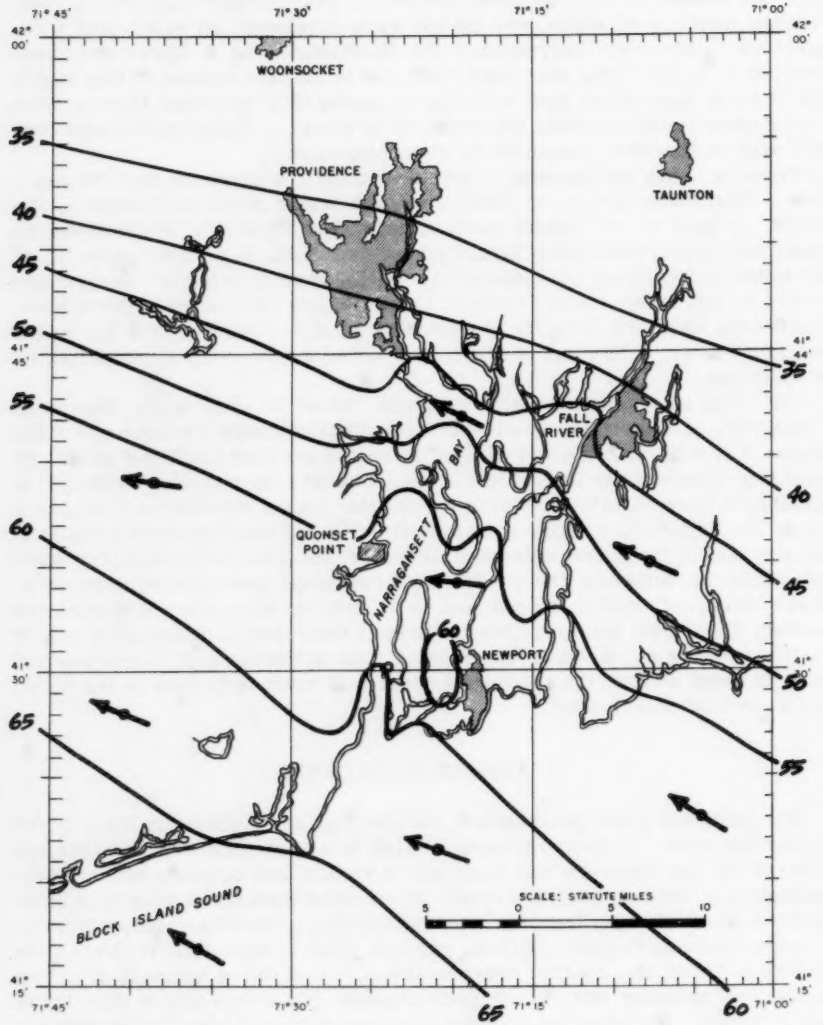

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FIG. 8.—SYNTHESIZED WIND FIELD (MPH) NARRAGANSETT BAY, SEPT. 14, 1944, 2100 EST

Another would be clearly "impossible" because it was far beyond anything that has been known to happen anywhere in the world. Meteorological consultants to engineers interpret that portion of the job something like this: First, narrow down these limits, bring them closer together. Second, put down numbers that represent the pooled judgment of a professional group as to the center of the range. Third, prepare a report that will not only give the engineer-user confidence that the job has been well done, but will also give him enough information to make his own final choice. The final choice and specification of a weather event for a design purpose is up to the design engineer, not only because he is the responsible official, but because he is in the

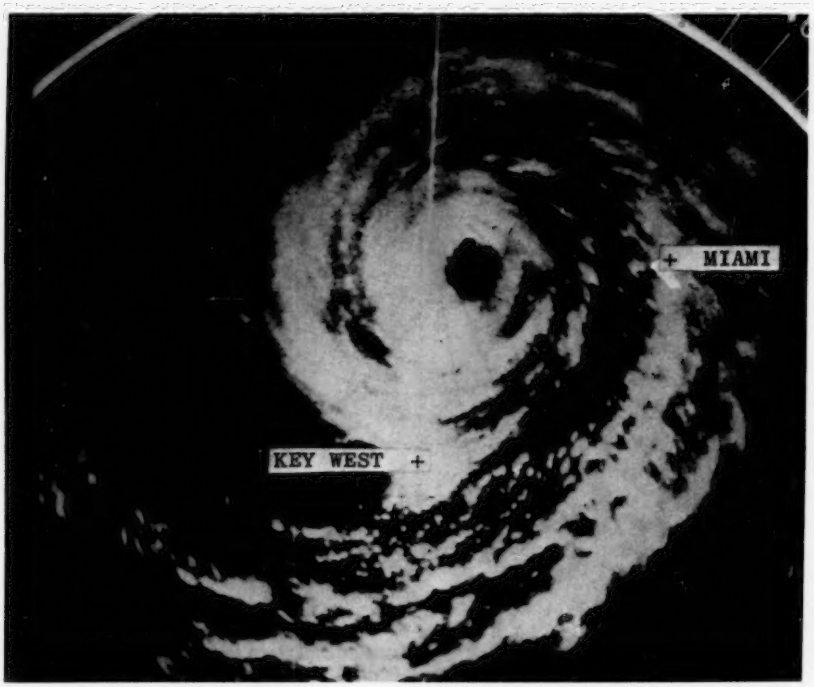

FIG. 9.—RADAR PHOTOGRAPH OF HURRICANE DONNA, SEPT. 10, 1960, 1340 GMT

best position to judge which of the categories of choice, frequent versus infrequent, and so forth, is the one that is appropriate to his project.

Of course once some design decisions have been made in a particular field of endeavor, communication becomes easier. The meteorological consultant can then, in effect, say if the acceptable risk for the current project is the same as for a group of prior projects, then the corresponding design storm has certain specifications. Thus, precedent helps standardize.

Some names applied to maximum hurricanes over coastal waters are "probable maximum hurricane" and "standard project hurricane." Only cur-

sory studies have been made of what the probable maximum hurricane in the New England area would be, as neither public policy nor the attitudes of individuals demand complete protection against an event of that severity or rarity. The probable maximum hurricane might be thought of as the worse the present climate is capable of producing, ocean water temperatures remaining the same; general storm tracks remaining the same, and so forth. But within this framework of conditions, other factors, both independent and interdependent, would work together to produce the most unfavorable combination.

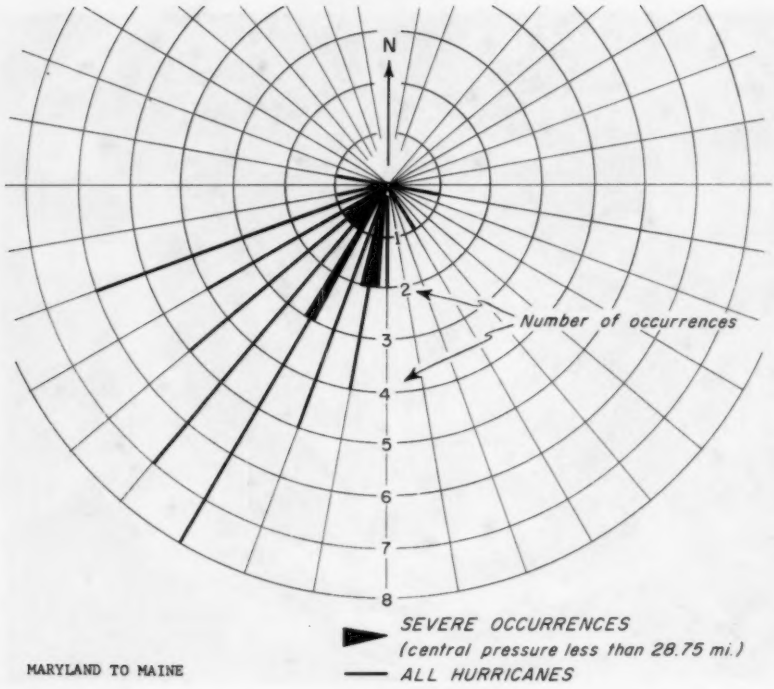

FIG. 10 —AZIMUTH DISTRIBUTION OF PATHS FOLLOWED BY HURRICANES (1887-1956)

Most of the work on maximized hurricanes in the New England area has been on a "standard project hurricane," or something rather comparable to it. The standard project hurricane lies between the maximum event of record and the probable maximum, and is closer to the maximum event of record. In a bulletin on standard project hurricanes[8] co-authored by the Hydromete-

[8] "Meteorological Considerations Pertinent to Standard Project Hurricane, Atlantic and Gulf Coasts of the United States," by H. Graham and D. N. Nunn, U. S. Weather Bur., Natl. Hurricane Research Proj. Report No. 33, 1960.

orological Section of the USWB and a representative of the Office of Chief of Engineers, United States Army, the primary purpose is stated as to

> represent a standard against which the degree of protection finally selected for a hurricane protection project may be judged and compared with protection provided in other localities.

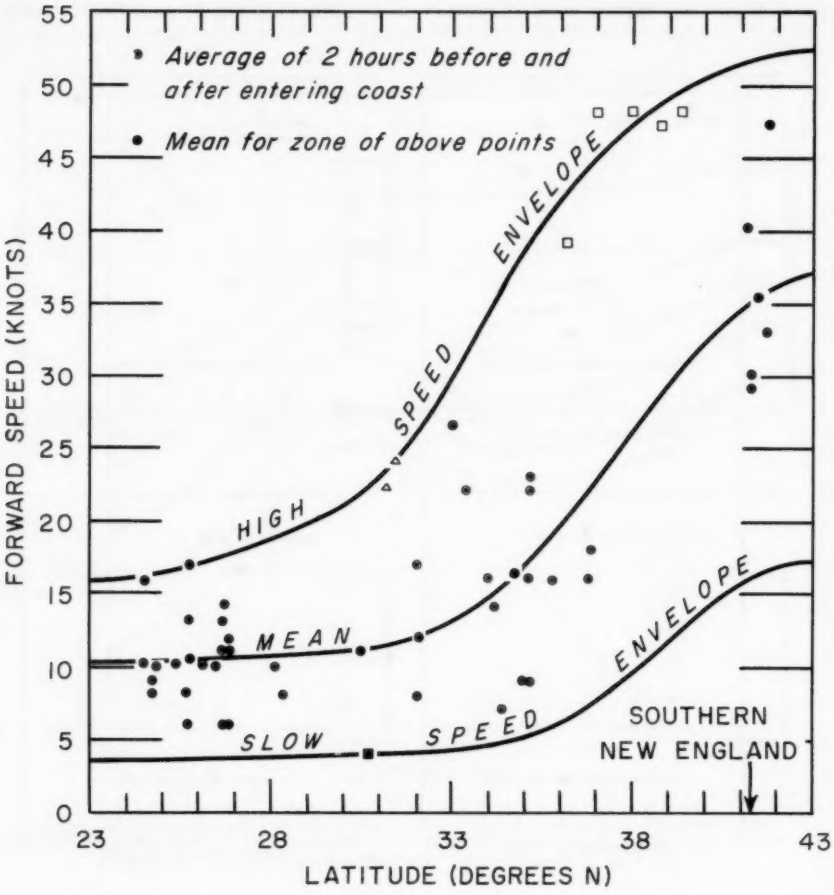

FIG. 11.—LATITUDINAL VARIATION OF RATE OF MOVEMENT OF HURRICANE CENTER ATLANTIC COAST U. S. (1900-1956)

In determining standard project criteria the basic procedures or goals are to envelop the characteristics of past hurricanes along the coast, work out variations from point to point that are climatologically reasonable, take into account inter-relationships between the variables such as radius of maximum

winds versus central pressure, and to extrapolate slightly beyond the record in a consistent fashion. Some data pertinent to New England will be presented herein from the standard project hurricane bulletin.

EXAMPLES OF MAXIMUM WIND FIELDS FOR DESIGN

The first question an engineer or official concerned with coastal protection asks might be, "from what direction will a hurricane approach this coastal

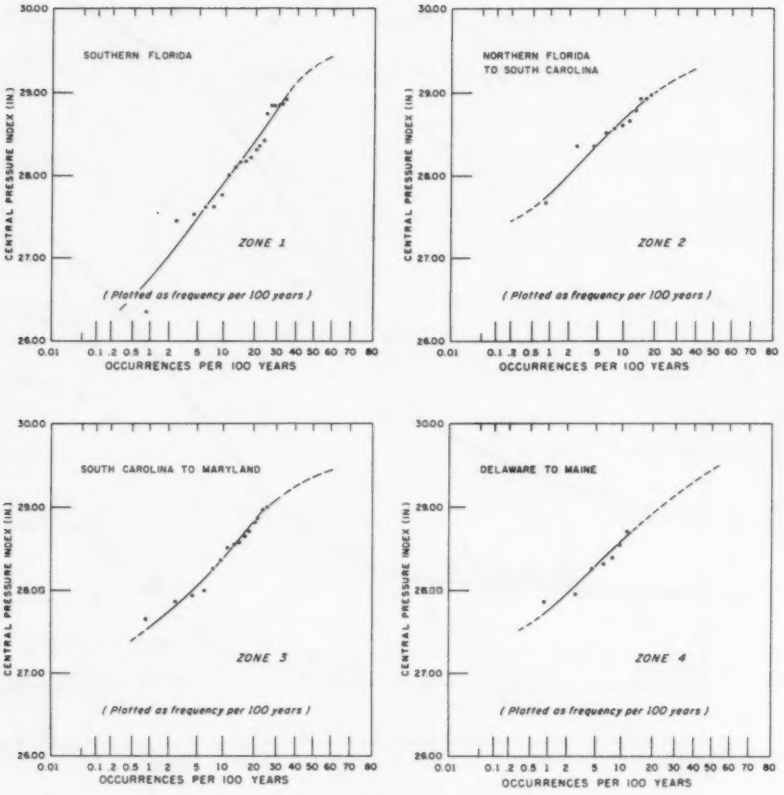

FIG. 12.—ACCUMULATED FREQUENCIES OF HURRICANE CENTRAL PRESSURE (1900-1956)

reach?" Fig. 10, from the standard project bulletin, shows the direction of past hurricanes in the reach from Maryland to Maine. From Fig. 10 one would conclude that from south to south-southwest is the most likely direction of approach to the New England shore for a severe hurricane, and that southeast or east should be accepted only on the basis of further study. From the southwest is a likely direction for a hurricane to approach, but the storms from this direction are likely to have had a prior track over land and to be weak.

A second important question involves the rapidity of the storm's approach. Involved are not only factors related to preparedness, such as time required for closing gates, but also physical relations between storm speed and reaction of the surge wave. Present views are that certain speeds of the storm favor greater dynamic build-up of the surge than other speeds. Fig. 11 is a graph of hurricane forward speed along the Atlantic Coast versus latitude. Investigation has revealed little or no correlation between hurricane speed and other characteristics; a severe hurricane might be expected to approach at any speed within the range of speeds enclosed by the envelopes in Fig. 11.

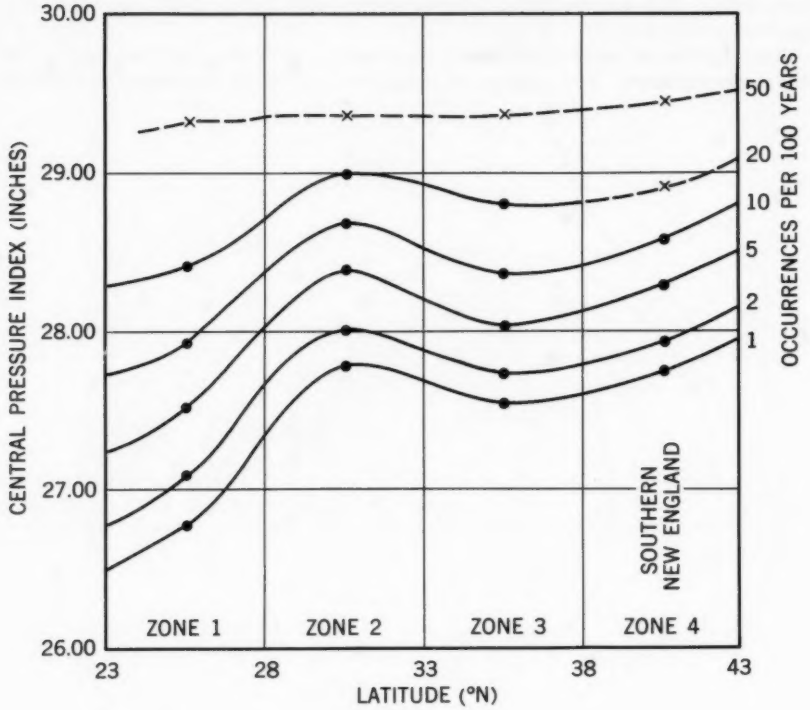

FIG. 13.—LATITUDINAL VARIATION OF AVERAGE FREQUENCY PER 100 YEARS OF HURRICANE CENTRAL PRESSURE, EAST COAST U. S. (1900-1956)

Thus a severe hurricane might be expected to approach the southern shore of New England at a speed anywhere between 10 knots and 50 knots. Theoretical considerations suggest that even faster speeds, while possible, would probably tend to result in a less severe rather than a more severe hurricane and, therefore, need not be given much weight for design purposes.

Next one might ask what intensities of hurricanes can be expected as indicated by the depression of the pressure at the center. Fig. 12 shows the central pressure of Atlantic hurricanes for a 57-yr period. It should be noted that many of the central pressures are estimates based on the various analysis

procedures rather than direct observations. The central pressure index is the value of the central pressure at the time the hurricane center crosses the coast. The latitudinal variation of the central pressure index is highlighted by constructing latitudinal graphs from the preceding four curves, as in Fig. 13. It is interesting to note that the central pressures that may be expected with various return periods at the southern New England coast are as severe as those expected in the reach of coast centered on Georgia. One would suppose that this is related to the shape of the coastline, with the New England area extending out to sea to intercept the characteristic northeastward track of hurricanes that have recurved. The standard project hurricane central pressure index, for geographical consistency, was taken from the once-per-hundred-years curve.

The radius of maximum winds was analyzed in the same manner as the other parameters. The radius of maximum winds is the distance from the

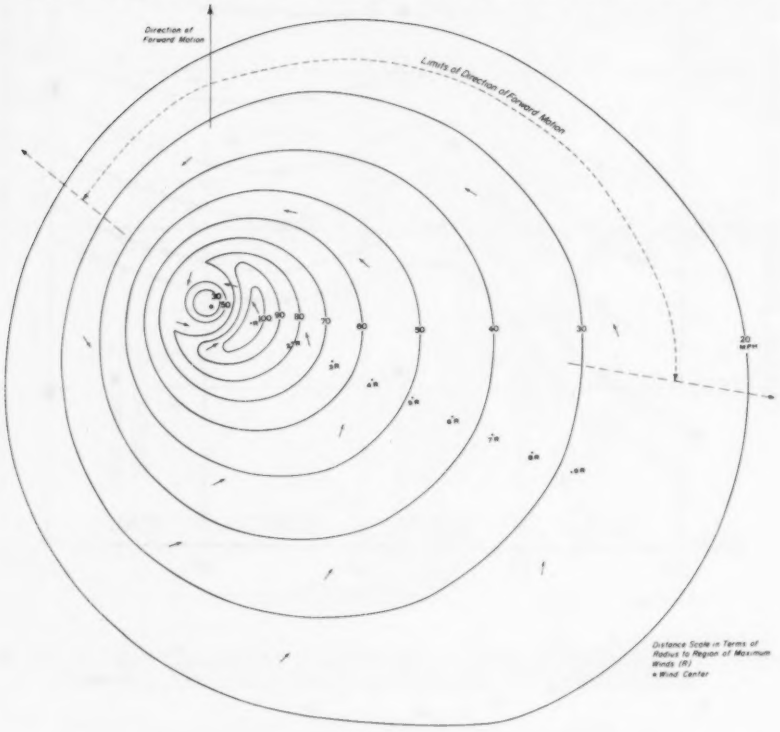
Direction of Forward Motion
Limits of Direction of Forward Motion
Distance Scale in Terms of Radius to Region of Maximum Winds (R)
Wind Center

FIG. 14.—STANDARD PROJECT HURRICANE WIND PATTERN FOR 40.5°N

storm center to the maximum speed band. The larger the radius of maximum winds, for a given maximum wind speed, the larger the area covered by high speeds and the greater the energy impacted to the water along a straight-line fetch. The radius of maximum winds for a severe hurricane increases with latitude. The standard project radius for southern New England is 50 nautical miles, corresponding to 38 nautical miles at Jacksonville, Fla., or only 12 nautical miles over the Florida Keys. These are envelopes of "observed" values (all obtained by indirect storm reconstruction), neglecting some weaker diffuse storms of large radius.

Combining standard project forward speed, central pressure, radius of maximum winds, and relations of speed to each of these variables yields a standard project wind field, shown in plan view in Fig. 14.

A somewhat different approach, that yields quite comparable results, was used for Narragansett Bay. The 1944 hurricane as it existed off of Cape

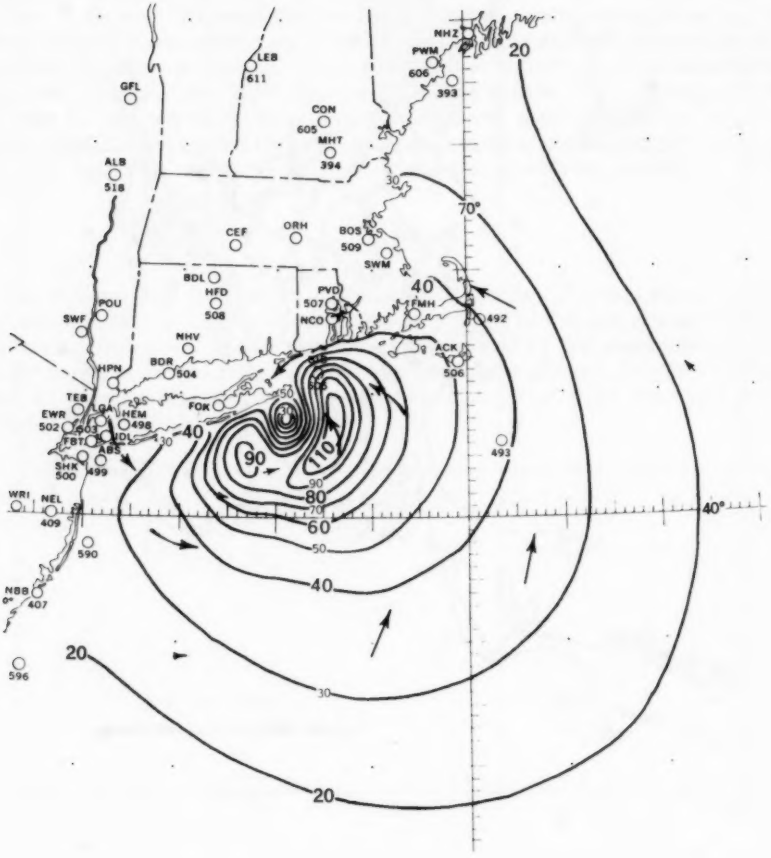

FIG. 15.—WIND FIELD (MPH) IN HYPOTHETICAL HURRICANE

Hatteras on the morning of September 14, 1944, is the most severe known hurricane (18th and 19th century hurricanes excluded for lack of data) close to the eastern seaboard of the United States. The storm actually capsized two modern Coast Guard cutters. It was agreed that this storm transposed to the

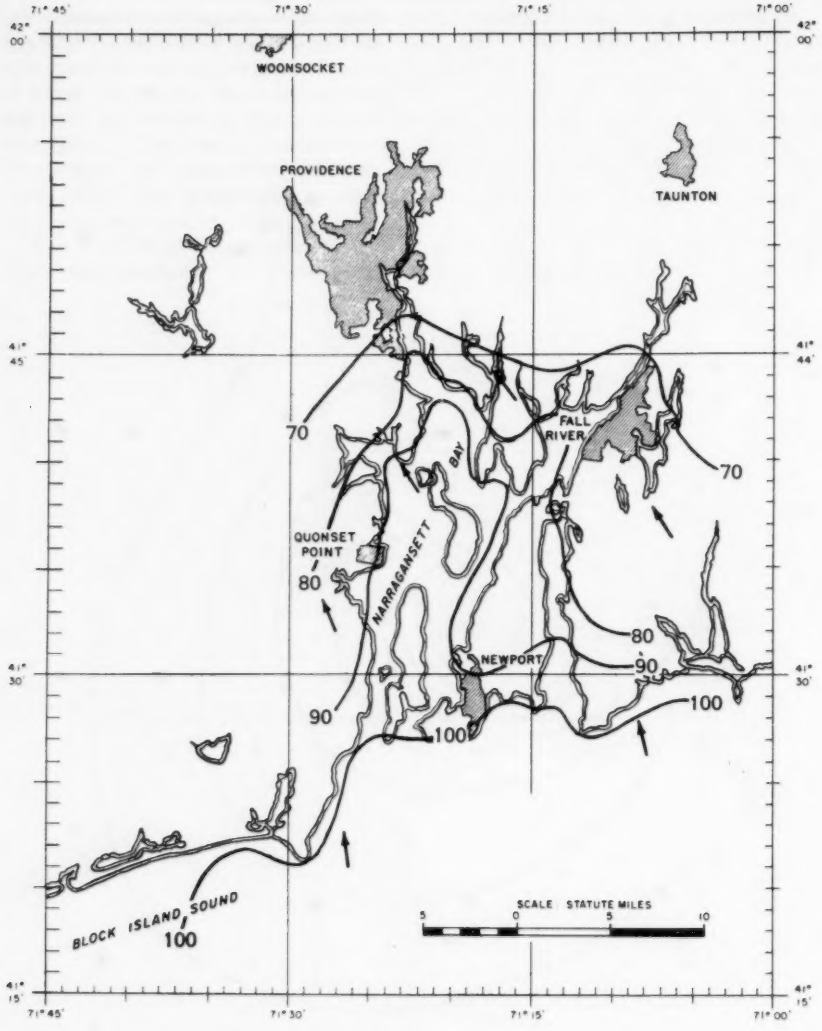

FIG. 16.—WIND FIELD (MPH) OVER NARRAGANSETT BAY IN HYPOTHETICAL HURRICANE

New England coast, would be appropriate for design in Narragansett Bay. This was done, with a minimum rate of decay from Cape Hatteras to New England. (The actual storm decayed faster before reaching New England on the night of September 14, 1944).

Fig. 15 shows one such hypothetical wind pattern modeled on the transposed 1944 storm. Fig. 16 shows the corresponding hypothetical wind pattern 1 hr later over Narragansett Bay.

A simple, but very significant, aspect of the application of a design hurricane wind field is the manner in which the computed meteorological tide is combined with the normal predicted tide. The arrival of the 1938 New England hurricane at high tide and, even more, the extraordinary winter disaster of the late 1950's in Holland, are reminders that the coincidence in time of an extraordinary storm with high tide is not to be dismissed. An example of a practical solution is the recommendation of the New England Division of the Corps of Engineers for the Fox Point hurricane barrier at Providence, R. I., that the design meteorological tide be combined with the average height of spring tide. This is more conservative than merely combining with average high tide but does not go to the point of combining with the maximum spring tide.

CONCLUSIONS

Hurricanes must be considered as part of the New England climate. Data derived by indirect analysis are shown for some of the major past storms.

Coastal protection against hurricanes is a matter for close consultation and cooperation between the meteorologist and the engineer. The two working together have developed a "standard project hurricane" to be used as a guide to design for the New England area.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

PNEUMATIC BREAKWATERS TO PROTECT DREDGES

By James L. Green,[1] F. ASCE

SYNOPSIS

A curtain of air bubbles from a submerged pipe will materially reduce heights of waves that are not longer than five times the depth. Tests of pneumatic breakwaters and models indicate their inability to protect dredges from long ocean swells off the New Jersey shore. A British air distributor might improve performance.

INTRODUCTION

The pneumatic breakwater consists of a submerged perforated pipe through which compressed air is forced. The pipe may be either suspended in the water or placed upon the ocean floor. As the compressed air is emitted into the water through the perforations, the air rises and a curtain of bubbles creates an upward motion of the air-water mixture. When this mixture reaches the surface, the air escapes and the flow of water branches out into two horizontal currents—one in the direction of the waves, and the other opposing them. The waves are attenuated (reduced in height) by a combination of the turbulence of the air-water mixture, and the movement of the newly created opposing current which causes premature breaking and consequent reduction in height and energy of the waves.

Note.—Discussion open until October 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 2, May, 1961.

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In a British patented air distributor, the air is released in intermittent bursts of large bubbles rather than steadily through perforations. The effectiveness of this breakwater is attributed by the patent owners, at least in large part, to the interference of the water stream generated by the bubbles with the orbital motion of the particles, that is characteristic of water waves. It may be that the special method of releasing the air makes the breakwater effective at the lower levels near the bottom, whereas breakwaters with jets have most of their effect concentrated in the surface current. This would tend to make the British distributor more effective than the jets in attenuating a wave when the length of the wave is more than twice the depth of the water.

The L/d- ratio (length of wave divided by depth of water) is the important indicator of the conditions under which a pneumatic breakwater is effective. In water-wave theory, when $L/d \leq 2$ the wave behaves as a "deep-water" wave, while for increasing L/d- ratios it begins to behave as a "shallow-water" wave. The pneumatic breakwater has been found most effective against "deep-water" waves; as the L/d- ratio increases beyond 2 the attenuation of the waves for a given power consumption drops off, and the breakwater rapidly becomes relatively ineffective for larger L/d- ratios, regardless of power consumption.

The relationship between the wave length, L, the wave period, T, and the depth, d, is given by

$$L = 5.12\, T^2 \tanh \frac{2\pi}{L/d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

For deep-water waves ($L/d \leq 2$) the function $\tanh 2\pi/(L/d)$ is nearly unity, and Eq. 1 becomes

$$L = 5.12\, T^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

For this study, when the wave is given in terms of its period, Eq. 2 has been used in all cases, for simplicity. This does not cause much error within the range of values of L/d with which the paper is concerned. For example, for $L/d = 5$, $\tanh 2\pi/(L/d) = 0.85$, and Eq. 2 gives an L/d- ratio that is about 17% too high.

If pipeline dredges could operate safely off shores exposed to the ocean, eroded beaches might be built up with sand from the sea bed. A study was made in the Department of Civil Engineering at Princeton University, Princeton, N. J. of the feasibility of using pneumatic breakwaters to protect dredges operating within about 1 mile of the shore of New Jersey in the area from Sandy Hook to Manasquan Inlet.

A height of 3 ft was taken as the greatest height of long ocean swells in which a pipeline dredge would be safe. For a pneumatic breakwater to provide practical protection it would have to insure that few, if any, waves of greater height would reach the dredging equipment. The study was then, essentially, one to determine whether available data on the performance of pneumatic

breakwaters would show that they could furnish this degree of protection for sufficiently long periods to permit economical operations.

REVIEW OF PREVIOUS STUDIES

Among the investigations that have been made model tests are the most frequent, but the first use of the pneumatic breakwater consisted of full-scale applications in the United States. In the 1930's there were some full-scale tests in Russia, and a number of full-scale tests have been performed in recent years (1952-1960) in England and in Japan.

Full-Scale Tests

Brasher's Pneumatic Breakwater.—The pneumatic breakwater came into being in the early 1900's when Philip Brasher obtained a patent on a method for attenuating waves by forcing compressed air through a submerged pipe.[2,3,4] One of Brasher's early tests was at Crotch Island, Maine (1908), where he installed a pneumatic breakwater to protect a wharf. Also in 1908, the pneumatic breakwater was used in salvaging the *U.S.S. Yankee* from sinking in heavy seas off the Massachusetts coast. In 1911 and 1912 Brasher installed his breakwater at Gloucester, Mass., where it was placed to insure shelter to shipping lying at a quarry wharf. The pneumatic breakwater was said to attenuate the waves remarkably well in all these trials, but data were not obtained on wave lengths, water depths, and wave heights. In 1915, Brasher built his most famous pneumatic breakwater at El Segundo, Calif., where it is credited with preventing an oil pier from being destroyed by the heavy winter storms. As in his earlier trials no data on wave lengths and water depth were taken.

Work in the U.S.S.R., 1930's.—In about 1930 there were several full-scale trials of the pneumatic breakwater in the Soviet Union.[5] Tests were made off the port of Mahatch-Kala on the Caspian Sea, in the port of Tuapse in the Black Sea, and at Vladivostok. The Mahatch-Kala tests gave some data on the installation of the breakwater, but like the other tests, no data on the L/d- ratio are available.

Dover, England, 1952.—In 1952, a full-scale pneumatic breakwater was tested at Dover, England.[6,7,8] The purpose of the installation was to protect the inner gate of a train ferry dock from the action of waves while the outer gate was being removed and repaired. The installation was temporary and operated for only a few months. It used the perforated-pipe method of releasing the air. There is uncertainty about how much this breakwater actually

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2 "Smashing Angry Seas with Bubbles of Compressed Air," by R. G. Skerret, Compressed Air Magazine, Vol. 26, January, 1921, pp. 9921-9926.

3 "The Brasher Air Breakwater," Engineer, Vol. 121, May 19, 1916, pp. 414 ff.

4 "The Brasher Air Breakwater," by Philip Brasher, Compressed Air Magazine, Vol. 20, 1915, p. 7523.

5 "The Compressed Air Breakwater," by I. A. Bogolepoff, Bulletin, Permanent International Assoc., Navg. Congresses, No. 23, January, 1937, pp. 101-125.

6 "Pneumatic and Similar Breakwaters," by J. T. Evans, Dock and Harbour Authority, Vol. 36, December, 1955, pp. 251-256.

7 "Pneumatic Breakwaters," by A. H. Laurie, Dock and Harbour Authority, Vol. 33, May, 1952, pp. 11-13.

8 "Pneumatic Breakwater at Dover," Dock and Harbour Authority, Vol. 33, December, 1952, p. 249.

accomplished in the way of quieting waves,[6] and it has not been possible for the writer to obtain data on the L/d- ratios and the corresponding attenuation of the waves.

Dover, England, 1956-1960.—The experience gained in the 1952 installation led to further research and development of the pneumatic breakwater at Dover.[9,10,11,12] A second full-scale test was performed in 1956 in the harbor of Dover, where waves varying from 8 ft to 10 ft in height were said to be reduced to half their height by using an improved air-supply system. Instead of having a uniform discharge of air from a submerged pipe, air containers were fastened to the submerged pipe, and the compressed air was released in pulses from these containers. The waves were interrupted as they rolled toward land and their heights reduced.

In 1957, following the 1956 trials, there was constructed at Dover a new pneumatic breakwater consisting of two parallel lines 100 ft apart, each 300 yd long, using the new method of air release with distributors spaced at 7-ft intervals. This breakwater stretched across the mouth of the inner basin in order to afford protection from easterly waves and swell. Six air compressors were used, each of 500 cfm capacity at 100 psi, but the compressors were modified to operate at 30 psi. For equivalent wave reduction, the air consumption was 5.1 cfm (0.45 hp) per yd of wave crest, as compared with 37 cfm (4.5 hp) in the 1952 equipment (using air jets).

It was observed that, "Normal waves whose height is less than 2 1/2 to 3 feet are unaffected by the curtain of eddies" set up by the breakwater. When the incoming waves exceeded this height the breakwater "established a 'defense in depth' of turbulent water through which a wave cannot pass without losing a substantial proportion of its original energy."

This equipment was reported[9] still in operation in early 1959. In April 1960, the equipment was salvaged and incorporated into new equipment. This new equipment was described as fully mobile, with its own built-in flotation system so that it could be raised quickly for maintenance or towing to a new location. The writer has been advised that in March, 1961, this equipment underwent sea trials off the southern coast of England.

The writer has not been able to obtain data on wave periods and L/d- ratios for any of the tests at Dover. In a subsequent section a development of probable L/d- ratios will be given.

First Full-Scale Test at Iwo Jima Islands, Japan, 1955.—In the winter of 1954-1955, full-scale tests were performed at the Iwo Jima Islands off Nagasaki.[13,14,15] The breakwater was a pipe 30 m long, 3 in. in diameter,

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9 "Pneumatic Breakwater Development in England," by William A. Heath, The Military Engineer, Vol. 51, March-April, 1959, pp. 104-106.

10 "The Pneumatic Breakwater," Pneumatic Breakwaters, Ltd., London, England, ca. 1957.

11 "Bubbles, Breakwaters, and Brasher," by G.R. Smith, Compressed Air Magazine, Vol. 64, No. 11, 1959, p. 20.

12 "A Wall of Bubbles Controls the Waves," Engineering News-Record, January 22, 1959, pp. 57-58.

13 "Pneumatic Breakwater Experiments in Japan," Dock and Harbour Authority, Vol. 38, August, 1957, pp. 137-140.

14 "On the Study of a Pneumatic Breakwater," by Michinori Kurihara, Part II, Reprinted in English from Reports of Research Inst. for Applied Mechanics, Kyushu Univ., Japan, Vol. IV, No. 16, 1956.

15 "Pneumatic Breakwater," by Michinori Kurihara, Part II, Trans. by K. Horikawa from Proceedings, Second Conf. on Coastal Engrg. in Japan, November, 1955; Wave Research Lab., Inst. of Engrg. Research, Univ. of Calif., Berkeley, Calif., 1958.

with 1.5 mm holes drilled at a rate of 35.7 holes per m, suspended in the sea to a depth of about 18 m below the lowest tidal level. The tidal range was about 3.5 m. This implies that the water was deeper than 18 m. Waves ranged in length up to 15 m. Consequently, the greatest L/d- ratio was less than about 0.84. The available air supply was 30 cu m per min. Due to the short length of the breakwater, the height of waves on its leeward side was affected not only by the reduction brought about by the breakwater but also by diffraction around the ends. This presented difficulties in interpreting the results.

An important preliminary conclusion reached by the investigators was that the reduction of wave heights by the breakwater was due not mainly to the surface current but rather to the turbulent viscosity accompanying the surface current, the surface current magnifying the effect of the turbulent viscosity.

Second Full-Scale Test at Ha-Jima Island, Japan, 1955-1956.—Additional tests, also off Nagasaki, were carried out during the winter of 1955-1956 as a continuation of the tests of the preceding year.[16,17,18] The breakwater was a pipe 60.5 m long, 5 in. in diameter, with 2 mm holes drilled at a rate of 47.6 holes per m, placed upon concrete blocks at the bottom of the sea, the pipe being about 16 m below the lowest tidal level. Waves ranged in length up to 30 m. Consequently, the greatest L/d- ratio was about 1.87. The available air supply was 150 cu m per min.

In one experiment, waves at least up to 25 m in length, with the pipe at a depth of 16.1 m (L/d about 1.55), were observed to be practically annihilated. In another experiment with a complicated wave pattern and the pipe at a depth of 15.6 m, the "significant" wave was judged to be 96.4 cm high and to have a period of 4.42 sec (deep-water length about 100 ft, L/d about 1.95). This wave was reduced by the breakwater by 31% of its original height. The wave fronts were inclined to the breakwater and diffraction took place around the ends. This led the experimenters to believe that the effect of the breakwater in reducing wave heights was better than indicated.

These experiments led to a confirmation of the preliminary conclusion of the first test, namely, that the eddy viscosity plays the important part in wave annihilation, with the surface current magnifying the effect of turbulent viscosity. The experimenters considered this also to be still a preliminary conclusion, and to need checking and correcting by watertank experiments and additional full-scale tests.

Third Full-Scale Test at Tsurumi Shipyard, Yokohama, Japan, 1958.—A test installation[19] was set up in August, 1957, but a chance to try it with long waves did not occur until the typhoon of July, 1958. The breakwater was placed to protect a fitting-out quay in the inner part of the shipyard basin. The depth of water near the quay was 6.5 m at low tide with a mean depth of 7.7 m.

The breakwater consisted of two parallel longitudinal pipes connected by short perforated pipes like the rungs of a ladder. The main pipes were 5 in.

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16 "Pneumatic Breakwater Trials," Dock and Harbour Authority, Vol. 36, April, 1956, p. 364.

17 "On the Study of a Pneumatic Breakwater," by Michinori Kurihara, Part III, Reprinted in English from Reports of Research Inst. for Applied Mechanics, Kyushu Univ., Japan, Vol. V, No. 20, 1957.

18 "Pneumatic Breakwater," by Michinori Kurihara, Part III, Trans. by K. Horikawa from Proceedings, Third Conf. on Coastal Engrg. in Japan, November, 1956, Wave Research Lab., Inst. of Engrg. Research, Univ. of Calif., Berkeley, Calif., 1958.

19 "Japanese Study of a Pneumatic Breakwater; Third Full-Scale Test under Natural Conditions During the 11th Typhoon (1958) at Tsurumi Shipyard, Yokohama, Japan," by Michinori Kurihara, Dock and Harbour Authority, Vol. 40, August, 1959, pp. 117-118.

in diameter, and the crossing pipes 3 in. in diameter, at 1-m intervals, with holes 2.5 mm in diameter drilled at a rate of 94 holes per m. Two ladders were made, one 1.5 m wide and the other 2.5 m, together with a straight single pipe for comparison. The straight pipe was 5 in. in diameter, drilled with 2.5 mm holes at 94 holes per m. It was found that the horizontal surface current induced by the ladder-type was 20% to 30% greater than the current induced by the straight pipe.

From dawn on July 23 the wind velocity increased, reaching 25 m per sec by 7:00 a.m. This velocity continued for about 2 hr, and then began to subside. The period of the waves became 4 sec (L = 25 m) at 6:00 a.m., and increased to a maximum of 5.2 sec (L = 42 m) at 9:00 a.m. The wave recorder was carried away by the storm at 7:30 a.m., and the observations were continued by eye. Up to about 100 cu m of air per min was supplied from 3:00 a.m. until 9:10 a.m. From the observations taken, the periods and heights of the "significant" waves were computed, and the results were published under the headings of Experiments Nos. 1, 2, and 3. Corrections were made to the observed wave reductions to exclude the estimated effect of diffraction around the ends of the breakwater.

In Experiment No. 1, where the highest waves occurred, the 2.5 m wide ladder, using 89 cu m of air per min, reduced 5.2 sec waves (L = 42 m) by 20% of their height. This took into account the effect on the water level of tide, wind, and barometric pressure. Taking the water depth as about 8 m, the L/d- ratio was about 5.

In Experment No. 2, the 1.5 m wide ladder at a depth of 7.5 m, using 115 cu m of air per min, reduced 3.42 sec waves (L = 18.5 m) by about 70% of their original height. The L/d- ratio was about 2.5.

In Experiment No. 3, the 2.5 m wide ladder at a depth of 7.6 m, using 74 cu m of air per min, reduced 4.0 sec waves (L = 25 m) by about 23% of their original height. The L/d- ratio was about 3.3.

Model Tests and Theoretical Investigation.—Since 1925 numerous model tests of the pneumatic breakwater have been made. Researchers have generally found that the relationship between wave length and water depth (L/d- ratio) is a critical factor in the effectiveness of a pneumatic breakwater. A resume of important model tests is given herewith.

Bogolepoff's Work, Central Scientific Research Institute for Water Transport, Leningrad, 1930's.—Bogolepoff conducted two sets of experiments, each with different apparatus.[5] Only the second set of experiments listed data on the wave lengths and water depths:

Apparatus:
Width of reservoir = 6.7 m
Length of reservoir = over 100 m
Depth of reservoir = maximum 3.5 m

With one pipe submerged to 2.5 m, the best results were obtained when a wave 22 cm high was completely destroyed. The wave length ranged from 1.5 m to 1.8 m, the most common being 1.75 m. The range of the L/d- ratio was, therefore, from 0.6 to 0.72. Thus, the experiments were all conducted with deep-water waves. No conclusions were drawn as to whether the air consumption was excessive.

S. Platzer, Sweden, 1938

Apparatus:
Water depth in wave tank = 2.80 ft
Tank width = 3.15 ft
Wave length = 3.1 ft

It will be seen that the L/d- ratio was 1.12, and deepwater wave conditions prevailed.[20] S. Platzer obtained a 90% attenuation of the waves in these tests, but the air consumption was said to be so large that further research in the field of pneumatic breakwaters was discouraged.

J. B. Schijf, Netherlands, 1940

Apparatus:
Width of tank = 2.40 ft
Range of wave lengths = 0.66 ft to 3.96 ft
Water depths varied from = 0.73 ft to 1.10 ft

The L/d- ratio ranged between 0.6 and 3.6. The maximum attenuation obtained was 75%. Schijf, like Platzer, concluded that too much air was required to make the pneumatic breakwater economical.[21] The air quantity was dependent on the L/d- ratio, the air required to attenuate the waves increasing with the L/d- ratio. The quantity of air, in Schijf's opinion, was too large for the pneumatic breakwater to be of any use.

G. I. Taylor, England, 1943.—A theory on the action of the pneumatic breakwater was developed in 1943 by G. I. Taylor in connection with unpublished work for the Admiralty.[22,23] His theory was modified and published in 1955. It attributed the reduction in wave heights to a surface current set up by the rising air bubbles. The current opposed the waves and resulted in some breaking and consequent loss of height.

J. H. Carr, California Institute of Technology, 1950.—J. H. Carr performed his experiments in a wave tank 60 ft long, 8 ft wide, and 12 in. deep.[24] Four submerged pipes were used for discharging the air. The experiments covered shallow-water waves with L/d- ratios up to 13. The wave length ranged from 2 ft to 13 ft, and the depth was constant at 1 ft.

For an L/d- ratio of 2, Carr obtained a maximum attenuation of 60%. At L/d = 7, the maximum attenuation was only 15%, and it remained at this value for the higher L/d- ratios up to 13.

Using Taylor's theory, Carr also computed the values of deep-water wave lengths ($L/d \leq 2$) which would be completely stopped by a certain air flow, and the corresponding power requirements,[25] as shown in Table 1.

Carr concluded that on the basis of this theory, practical application of the pneumatic breakwater against deep-water waves was limited to those of shorter period than 5 sec or 6 sec.

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20 "Pneumatiska Vagbrytare," by Sven Platzer, Teknisk Tidskrift, Vol. 68, September, 1938, pp. 105-109.

21 "The Annihilation of Waves Through Injection of Air," by J. B. Schijf, De Ingenieur, Vol. 55, 1940, pp. 121-125.

22 "The Action of a Surface Current Used as a Breakwater," by G. I. Taylor, Proceedings, Royal Society of London, Vol. 321 A, 1955, pp. 466-478.

23 "Note on the Possibility of Stopping Sea Waves by Means of a Curtain of Bubbles," by G. I. Taylor, British Admiralty Report, ATR Misc. 1259, 1943.

24 "Mobile Breakwater Studies," by John H. Carr, Report No. N-64.2, Hydrodynamics Labs., Hydr. Structures Div., Calif. Inst. of Tech., Pasadena, Calif., December, 1950.

25 Ibid., p. 20.

J. T. Evans, Great Britain, 1954

Apparatus:
Wave tank width = 4.0 ft
Wave tank depth = 4.0 ft
Wave tank length = 62.0 ft

Wave lengths varied from 2 ft to 12 ft. Using a depth of 3 ft, the L/d- ratio ranged from 0.67 to 4.

When the L/d- ratio was less than or equal to 2, the horsepower per foot of crest required to stop a wave of length L varied from $0.29\ L^{2.5}\ 10^{-4}$ to $1.31\ L^{2.5}\ 10^{-4}$. As the L/d- ratio was increased from 2 to 4, the horsepower required to attenuate the waves increased from 1 to 4 times as much as these values.[6] Evans concluded that the complete calming of waves longer than 200 ft would not be economical or even possible.

TABLE 1.—AIR FLOW AND POWER REQUIREMENTS TO STOP WAVES OF CERTAIN LENGTHS (PERIODS)[24]

Air flow, in cubic feet per minute (1)	Wave length, L in feet (2)	Wave period, T, in seconds (3)	Power requirement, in horsepower per foot of crest (4)
300	100	4.4	20
4,000	300	7.7	260
24,000	600	10.8	1575

D. Linke, and W. Hensen, Franzius Institute, Germany, 1955

Apparatus:
Tank width = 2.2 m
Tank length = 112 m
Water depth = between 1.15 m and 1.17 m
Number of submerged pipes = from 1 to 5

The range of L/d was between about 1.7 and 7.0; and a wave attenuation between 10% and 73% was obtained.[26]

St. Anthony Falls Hydraulic Laboratory Experiments, University of Minnesota, 1955

Apparatus:
Tank width = 6 in.
Tank depth = 16 in.
Tank length = 40 ft

When the L/d- ratio was less than 2, the pneumatic breakwater was highly effective, but for larger values of L/d the effectiveness declined.[27] Table 2 gives the largest L/d- ratio for which the stated attenuation was obtained.

[26] "Model Test with Pneumatic Breakwaters," by Walter Hensen, Dock and Harbour Authority, Vol. 36, June, 1955, pp. 57–61.

[27] "Experimental Studies of Pneumatic and Hydraulic Breakwaters," by J. M. Wetzel, Proj. Report No. 46, St. Anthony Falls Hydr. Lab., Univ. of Minn., Minneapolis, Minn., May, 1955.

St. Anthony Falls Hydraulic Laboratory Experiments, University of Minnesota, 1959.—In carrying out the experiments in 1959, two wave tanks were used.[28] The small tank had the dimensions:

Width = 2 ft
Depth = 1 ft 3 in.
Length = 50 ft

The large tank:

Width = 9 ft
Depth = 6 ft
Length = 253 ft

The range of L/d- values in the small tank was from 1.2 to 4.2, and in the large tank from 1.2 to 2.4.

For a given L/d- ratio the attenuation increased with the air discharge. Table 3 gives the maximum attenuation obtained for each L/d- ratio.

The difference in the attenuation of waves is more marked in the small-tank tests, where an L/d- ratio of 1.2 gave a maximum of 90% attenuation, compared to an L/d = 4.2, which gave only a 30% attenuation.

TABLE 2.—ATTENUATION OF WAVES WITH RESPECT TO THE L/d RATIO AT ST. ANTHONY FALLS, 1955[27]

L/d	Attenuation, in %
1.65	90
2.00	70
2.37	50
3.00	37

TABLE 3.—ATTENUATION OF WAVES WITH RESPECT TO THE L/d- RATIO AT ST. ANTHONY FALLS, 1959[28]

Maximum Attenuation, in %	L/d- Ratio	Maximum Attenuation, in %	L/d- Ratio
(a) Small tank		(b) Large tank	
90	1.2	80	1.2
85	2.4	75	1.8
65	3.5	65	2.4
30	4.2		

Beach Erosion Board, 1959.—Tests of the pneumatic breakwater were conducted in the large wave tank at the Beach Erosion Board, Corps of Engineers, United States Army, by the Transportation Corps; the data were obtained by the writer in a personal communication. The depth of water was 15 ft.

Up to 70% attenuation was obtained of a wave with a period of 2.61 sec (L = 35 ft). The L/d- ratio was approximately 2.3. For a longer period of 7.87 sec (L = 317 ft) the attenuation was very small; the L/d- ratio was 21. An

28 "Experimental Studies of Pneumatic and Hydraulic Breakwaters," by L. G. Straub, C. E. Bowers, and J. S. Tarapore, Tech. Report No. 25, Series B., St. Anthony Falls Hydr., Lab., Univ. of Minn., Minneapolis, Minn., August, 1959.

attenuation of about 25% was obtained for a 3.75-sec wave (L = 72 ft, L/d = 4.8); and less than 10% attenuation was obtained for a 5.6-sec wave (L = 160 ft, L/d = 10.7).

It should be noted that the air supply was not adequate, and somewhat better results might have been obtained with more air.

Summary of Previous Studies.—The studies have shown that the L/d-ratio is the important indicator of the usefulness of a pneumatic breakwater. In Fig. 1 the results obtained by the various investigators are summarized in the form of a plot of wave attenuation versus L/d- ratio. On this plot each investigator's greatest attenuation is shown for each of his various L/d- ratios. Because attenuation increases with power consumption, the greatest attenuation for each L/d- ratio is, in general, the one which required the greatest power consumption for that L/d- ratio. By taking an approximate mean line through the plotted points, giving somewhat less weight to the tests in Japan and those at the Beach Erosion Board because both were done with limited air supply,

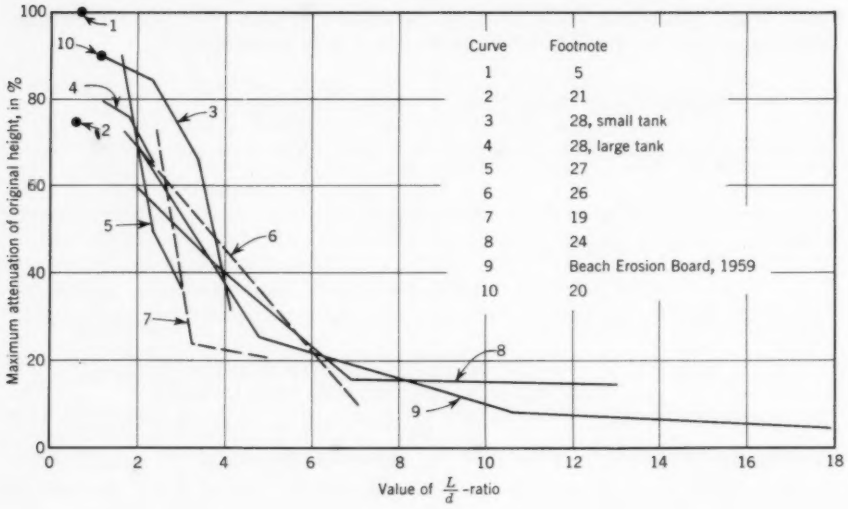

FIG. 1

Table 4 was obtained. Table 4 gives a fair idea of the most optimistic results to be expected from a pneumatic breakwater with unlimited air supply on the basis of previous studies, predominantly with models.

FEASIBILITY OF USING PNEUMATIC BREAKWATERS TO PROTECT DREDGES OFF THE NEW JERSEY SHORE FROM SANDY HOOK TO MANASQUAN INLET

Coast and Geodetic Survey, United States Department of Commerce (USGS), Chart 824 covers the area off the New Jersey Shore from Sandy Hook

to Manasquan Inlet.[29] The range of the tide is about 4 ft. A depth of 52 ft was taken as representative of conditions at mean tide level about 1 mile from the shore, where the breakwater was assumed to be located. This depth is a little high in order to offset, to some extent, the fact that L/d- ratios were computed with deep-water wave lengths.

Statistics on waves to be expected in this area, based upon "hindcasting," have been published by the Beach Erosion Board.[30] The applicable data are those for Station C off New York harbor and these tables show monthly statistics on wave conditions at this station over the 3-yr period from 1948 to 1950. The data are given in the form of wave periods, wave heights, and the hours of duration. These are "significant" waves (symbol $H_{1/3}$); a "significant" wave being one which has the period of the predominating waves and a height that is the average of the higher one-third of the predominating waves. The heights in the tables are for deep-water conditions, and therefore, the heights of waves in 52 ft of water will be somewhat lower than those given. However, the given data are used because the correction would be relatively small (about 1% for L/d = 2, and about 8% for L/d = 5).

In Beach Erosion Board Technical Report No. 4, a method is presented for estimating the frequency of occurrence of the higher waves in a wave train of a given "significant" height.[31] By this method it is found that if the "significant" height is 2 ft, the height which will not be exceeded 99% of the time is

TABLE 4.—ATTENUATION OF WAVE HEIGHT BY PNEUMATIC BREAKWATER WITH UNLIMITED AIR SUPPLY, FROM FIG. 1

$\frac{L}{d}$ - ratio	1	2 to 2.5	3	4	5	7	> 10
Attenuation, in %	90	70	50	40	25	15	< 15

about 3.4 ft. Similarly, if the "significant" height is 3 ft, the height not exceeded 99% of the time is about 4.8 ft. In order for the breakwater to reduce these greater heights to the "significant" height, the reductions obtained by the breakwater would have to be about 41% and 38%, respectively. These are about the most optimistic attenuations that could be expected when L/d = 4.0, and are greater than can be expected for larger L/d- ratios, as seen in Table 4. It is then reasonable to conclude that for "significant" heights of 2 ft to 3 ft and L/d- ratios of 4.0 or less the breakwater might be expected to insure that few waves would emerge on the leeward side with heights greater than the "significant" height.

For a "significant" height of 4 ft, the height not exceeded 99% of the time is about 6.5 ft. To reduce it to 3.0 ft would require an attenuation by the

[29] "Approach to Intracoastal Waterway, Sandy Hook Bay to Manasquan Inlet," Chart 824, 3rd Ed., Coast and Geodetic Survey, U. S. Dept. of Commerce, Washington, D. C., September 21, 1959.

[30] "North Atlantic Coast Wave Statistics Hindcast by Bretschneider—Revised Sverdrup-Munk Method, Tech. Memorandum No. 55, Beach Erosion Bd., U. S. Army Corps of Engrs., U. S. Govt. Printing Office, Washington, D. C., November, 1954.

[31] "Shore Protection, Planning and Design," Tech. Report No. 4, Beach Erosion Bd., U. S. Army Corps of Engrs., U. S. Govt. Printing Office, Washington D. C., June, 1954, Revised, August, 1957, p. 18.

breakwater of about 54%, more than can be expected for any L/d- ratio greater than 3.

In the light of these considerations, it was concluded that a reasonable indication of the breakwater's potential value would be the percentage of the time when the "significant" wave height is 3 ft or less and L/d is 4.0 or 5.0 or less. For each of these L/d- ratios analyses were made for "significant" wave heights of both 2 ft or less and 3 ft or less. The results are shown in Table 5.[30]

The hours of occurrence of calm sea, or of waves of "significant" heights of 0.5 ft to 2 ft, or 0.5 ft to 3 ft, were, of course, not all consecutive, or even necessarily consecutive for any particular period of time. The dredging operations would, therefore, probably require the protection of a breakwater even during calm hours as insurance against sudden swells. This being the case, the breakwater would be of some value for the percentages of time shown in Cols. 4, 7, 10, and 13 of Table 5. However, its value is dependent on an ability to take advantage of the hours of calm or or relatively low swells and to remove the equipment to safety during periods of high swells. Even during the months when the waves are lowest, the statistics show many hours when waves occur that the breakwater cannot be expected to reduce to acceptable heights, and some of these hours may occur at any time. Consequently, operations would likely suffer interruptions and damage to the equipment unless there were a reliable service to warn of approaching high swells. For these reasons, it does not seem that the breakwater is likely to be of practical advantage to dredging operations off the New Jersey shore unless it can be improved to the point where it will reduce to 2 ft or 3 ft not only waves in low ranges of "significant" height but also the great majority of all waves that are likely to occur during extended periods.

The degree to which a single pneumatic breakwater in its present (1961) state of development could definitely not be expected to do this is indicated by the differences between 100% and the percentages in Cols 4, 7, 10, and 13 of Table 5. On the basis of the data in these tables the breakwater might make dredging possible the highest percentage of the time in the summer months of June through August. For these months, using the data in Table 5(d), a single breakwater would definitely be inadequate the following percentages of the time:

June	July	August
29%	12%	38%

It will be noted that if any one of Tables 5 (a) to (c) had been used instead of Table 5(d), the result would not have been greatly different. Thus, the single breakwater does not appear able to insure uninterrupted safe operations during the most favorable months.

The possibility of using multiple breakwaters was considered:

In June the predominant longer waves range up through periods of 10 sec to 12 sec (L/d = 10 to 14) with heights up to 6 ft to 8 ft. There are a few waves of this L/d- range with heights up to 10 ft to 12 ft, and a few 12 sec to 14 sec (L/d = 14 to 19) waves with heights in the 4 ft to 6 ft range.

In July, the calmest month, the predominant longer waves again range up through periods of 10 sec to 12 sec (L/d = 10 to 14) but with only a few heights up to 4 ft to 6 ft. There are a few waves in the 12 sec to 14 sec (L/d = 14 to 19) range but with heights in the 2 ft to 4 ft range.

TABLE 5.—HOURS OF OCCURRENCE OF SIGNIFICANT WAVES[a]

Month	Hours of Calm Sea[b]	Hours of Significant Waves[c]	[(Col. 2 + Col. 3) ÷ Hours in Month] x 100	Hours of Calm Sea[b]	Hours of Significant Waves[d]	[(Col. 5 + Col. 6) ÷ Hours in Month] x 100	Hours of Calm Sea[b]	Hours of Significant Waves[c]	[(Col. 8 + Col. 9) ÷ Hours in Month] x 100	Hours of Calm Sea[b]	Hours of Significant Waves[d]	[(Col. 11 + Col. 12) ÷ Hours in Month] x 100
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
	(a) $L/d < 4.0$; 0 ft $< H_{1/3} < 2$ ft			(b) $L/d < 4.0$; $0 < H_{1/3} < 3$ ft			(c) $L/d < 5.0$; $D < H_{1/3} < 2$ ft			(d) $L/d < 5.0$; $0 < H_{1/3} < 3$ ft		
January	250	1	34	250	4	34	250	33	38	250	41	39
February	252	8	38	252	15	39	252	29	41	252	37	43
March	285	7	38	285	7	39	285	33	43	285	36	43
April	320	0	43	320	1	43	320	16	45	320	20	46
May	266	13	38	266	25	39	266	39	41	266	52	43
June	483	8	68	483	11	69	483	23	70	483	28	71
July	595	15	82	595	26	84	595	50	87	595	62	88
August	400	21	57	400	35	58	400	41	59	400	59	62
September	219	15	33	219	21	33	219	39	36	219	49	37
October	108	20	17	108	33	19	108	89	26	108	111	29
November	249	1	35	249	5	35	249	47	41	249	54	42
December	241	0	32	241	2	33	241	23	35	241	26	36
Full Year	3,668	109	43.1	3,668	185	43.9	3,668	462	47.1	3,668	575	48.3

[a] From footnote 31. Entries are averages for a 3-yr period. [b] "Significant" height of less than 0.5 ft. [c] "Significant" height of from 0.5 ft to 2.0 ft. [d] "Significant" height of from 0.5 ft to 3.0 ft.

In August, however, the longest waves increase up to 18 sec to 20 sec (L/d = 32 to 40) with heights up to 12 ft to 14 ft not uncommon, and one year 12 sec to 14 sec waves were 25 ft to 30 ft high. Conditions in this month thus appear to be too severe for any pneumatic breakwater in its present state of development.

In Fig. 1 it will be seen that the best attenuation in any experiment for L/d = 10 was about 15%. This would be inadequate even for the months of June and July. However, if the breakwater could be improved to effect 40% to 50% attenuations at L/d- ratios up to about 20, it could have possibilities for both of these months. Two or three breakwaters might be placed across the path of the incoming waves, the breakwaters being separated by enough distance, say 1,000 ft or more, to act separately. If each of these breakwaters attenuated a 12-ft wave by 40%, it would be only 2.6 ft high after passing the last one. However, with present prospects of only 15% or less attenuation at best, the wave could most likely not be reduced by three breakwaters to a height below 7.5 ft.

If pneumatic-breakwater equipment were readily mobile it could not only be placed in the form of successive breakwaters, each attenuating the oncoming waves, but it could also be moved to new locations as the direction of the waves shifted, and the length and alinement of each breakwater could be altered as needed to lessen diffraction around the ends. For the lower waves, only one breakwater might suffice, and with all equipment controlled from a central location, the second and third breakwaters could be operated only when needed. Similarly, the air supply could be regulated to furnish only as much as required, with the full capacity needed only for the high L/d- ratios.

However, it appears that in any event a large amount of equipment will be required for dredging operations in a location exposed directly to ocean waves. This is said with the present (1961) type of pipeline dredging equipment in mind. Naturally, equipment that could operate in higher swells would demand less protection from the breakwater.

The British air distribution has one feature not incorporated into any other models or full-scale applications known to the writer, namely the intermittent air distributors. As stated previously there are some indications that this equipment may be specially effective for large L/d- ratios (say greater than 5), but until more specific data on its performance are available this cannot be determined.

As a check, a second analysis was based on observations of surf made under a cooperative program between the United States Coast Guard and the Beach Erosion Board.[32] The data applicable to this paper are for Monmouth Beach Lifeboat Station, N. J., that is near Station C described in a publication by the Beach Erosion Board.[30] Data are presented by months for the 3-yr period from October 1954 to October 1957, in the form of heights of "significant" waves and percentages of time when they occurred, together with monthly maximum, minimum, and average wave periods for the month. The period corresponding to each range of wave height is not given. Each observation included the time, in seconds, for ten complete breakers, and a visual estimate of the "significant" height of the surf.

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32 "Surf Statistics for the Coasts of the United States," Tech. Memorandum No. 108, Beach Erosion Bd., U. S. Army Corps of Engrs., U. S. Govt. Printing Office, Washington, D. C., November, 1958.

Table 6 was constructed after the deep-water wave lengths had been computed from the published wave periods, assuming no material change in the wave period between deep water and the observation station.

TABLE 6.—OCCURRENCE OF SURF AT MONMOUTH BEACH, N. J. (AVERAGES FOR A 3-YR PERIOD, EXCEPT AS NOTED). BASED ON SURF STATISTICS OF BEACH EROSION BOARD[32]

Month	Occurrence of surf, in % of time			L/d maximum for period average for period minimum for period	% of hours of significant height from 0 to 3 ft[a]
	"Significant" height, in feet				
	0-2	0-3	3 +		
(1)	(2)	(3)	(4)	(5)	(6)
January	59.9	78.2	21.8	7.8 4.8 2.6	78
February	53.0	74.0	26.0	9.8 5.2 2.2	74
March	44.0	64.8	35.2	8.5 4.8 2.0	65
April	32.5	59.2	40.8	8.3 4.7 2.1	59
May	54.9	74.8	25.2	7.3 4.8 1.6	75
June	49.0	71.0	29.0	7.1 4.6 1.1	71
July	46.3	67.5	32.5	8.0 4.8 2.5	68
August	28.0	54.7	45.3	11.7 4.7 1.6	55
September	26.0	55.7	44.3	9.4 4.6 2.5	56
October	35.5	61.3	38.7	11.9 4.6 0.7	61
November	41.4	67.0	33.0	9.1 4.7 1.6	67
December	66.6	77.5	22.5	7.8 4.8 2.6	78

[a] $L/d < 5.0$; $0 < H_{1/3} < 3$ ft.

Data taken from Table 6, Cols. 2 through 5 have been used for Table 6, Col. 6 which is arranged for comparison with Table 5(d). In Table 5(d) the "significant" heights are for deep-water conditions, and in Table 6, Col. 6 they are observed "significant" heights of surf. It will be noted that the outstanding difference between Table 5(d) and Table 6, Col. 6 is that in the former the waves in the "significant" height range of 0 to 3 ft occur markedly more often in June, July, and August than in other months, whereas in Table 6, Col. 6 occurrence of waves in this range is much more nearly the same throughout the year, with the occurrences for the summer months fairly comparable in the two tabulations. This is probably because the waves lost a considerable percentage of their deep-water height before reaching the observation station, that would have the effect of decreasing the range of heights and increasing the frequency of occurrence of the lower heights.

In the absence of a better correlation between the L/d- ratios and the height ranges, it seemed reasonable to assume that the L/d- ratios for the 0 to 3 ft range were less than 5, which will be seen in Table 6 to be slightly larger than the average L/d- ratio that is nearly constant throughout all months.

The data in Table 6, Col. 6 lead to no more favorable indication of the usefulness of the pneumatic breakwater in its present state of development, because in all months there is a large percentage of the time (100% minus the percentages in Table 6, Col. 6) when the breakwater could not be expected to insure safe operations. The percentages of time in June and July when the breakwater would be inadequate are around the same if taken from Table 5(d) or from Table 6, Col. 6. In the other months, conditions are likely to be more unfavorable for the breakwater than Table 6, Col. 6 indicates, because the 0 to 2 ft and 0 to 3 ft ranges of "significant" height may include a considerable number of waves which had materially greater heights about 1 mile off shore, where it has been assumed the breakwater would be located.

BRITISH EQUIPMENT

Because L/d- ratios for the waves encountered by the British equipment at Dover, England, were not available, some estimate of the L/d- ratios will be attempted.

Evans[6] has described the 1952 breakwater at Dover as being 75 ft from the railway dock gate in the inner harbor. He says that the channel, under which the breakwater lay, was 80 ft wide and 40 ft to 50 ft deep, and that the waves were reported to be 120 ft to 150 ft long and about 4 ft high. These figures give an L/d- ratio between 2.4 and 3.75.

Evans goes on to say that waves longer than 250 ft seldom occur on the coasts of the British Isles except those exposed to Atlantic gales, with few reports of waves longer than 200 ft. Data supplied by the United States Navy Hydrographic Office and abstracted to obtain Table 7 indicate that waves longer than 250 ft occur on the south and east coasts of England from 3% to 10% of the time, depending on the season.

To reach the pneumatic breakwater at the entrance to the inner harbor, waves had to pass through the opening in the permanent outer breakwater, with some change in length and marked reduction in height. It seems reasonable to estimate that the great majority of waves reaching the pneumatic breakwater at Dover did not have L/d- ratios over 5, with most of the ratios well below

that value. This is the same range of values for which other investigators (Fig. 1) have obtained considerable reductions in wave heights. The important question, whether the breakwater at Dover effected greater reductions in this L/d- range, or in a higher range, than were obtained by others, could not be answered by the writer for lack of data.

There is some reason to believe that the answer could be affirmative. While discussing the tests at the Franzius Institute in Germany, A. H. Laurie has explained how part of a waves's energy may pass under the zone of effectiveness of the breakwater, which in those tests appeared not to reach down much more than one-third of the depth.[33] Taylor has deduced that this is about the

TABLE 7.—OCCURRENCE OF WAVES ON THE COASTS OF ENGLAND WITH HEIGHTS GREATER THAN 6 FT[a]

Months	Percentage Longer Than					
	250 ft (7 sec)	400 ft (9 sec)	600 ft (11 sec)	250 ft (7 sec)	400 ft (9 sec)	600 ft (11 sec)
	(a) South and east coasts			(b) South and west coasts		
January, February, March	3	1	0	15	7	3
April, May, June	3	1	0	3	2	1[b]
July, August, September	8	4	0	7	3	1
October, November, December	10	5	0	6	3	1

[a] Based on data from the United States Navy Hydrographic Office. [b] Approximately.

depth to which the surface current extends.[22] Now it is characteristic of water waves that if the L/d- ratio is small, most of the energy of the wave is in the same higher levels of the water as the surface current, and, therefore, the latter would be expected to be quite effective in damping the wave. However, if the L/d- ratio is over 2, an increasing amount of energy is in the lower levels of the water. This would explain the decreased effectiveness of the pneumatic breakwater for larger L/d- ratios.

The same limitation on the depth at which a pneumatic breakwater is effective probably applies to all of the tests that have been mentioned herein, except possibly those at Dover since 1955, because all others used essentially the same method of releasing the air in jets from holes drilled in the air pipe. The special feature of the British equipment is that the air goes first into the rows of cylindrical containers on the bottom, and then emerges from them in sudden, intermittent bursts, at time intervals controlled from the shore. These

[33] "The German Experiments on Pneumatic Breakwaters," by A. H. Laurie, Dock and Harbour Authority, Vol. 36, June, 1955, pp. 61-64.

large bursts of air are said to create vortices which interrupt the orbital motion of a wave and cause it partly to destroy itself.[10] This distinctive feature of vortices near the bottom may have the effect of starting dissipation of the wave's energy at greater depths than do other breakwaters which release the air from jets.

This explanation of how the L/d- ratio affects the action of the breakwater is the most plausible that the writer has found and, if this British method or releasing the air does interfere materially with the wave motion at greater depths, present indications of the value of the pneumatic breakwater may have to be modified when data are available on the wave attenuation obtained for various L/d- ratios by the latest British equipment. If the equipment should be able to effect large attenuations (for example, up to 40%) for high L/d- ratios (say up to 10), then it might be able to satisfy, for extended periods of time, the criterion that few, if any, waves over 3 ft high would reach dredging equipment protected by it on the open sea.

If this could be done, the matter of power consumption would have to be considered more closely, in the interest of economy, than has been necessary for present purposes. The 1952 British equipment, which used the jet method of releasing the air, required 37 cfm per yd of wave crest, while in 1958 the equipment using the intermittent air release method required only 5.1 cfm per yd of crest.[9,10] The latest equipment might have further improved efficiency, but at large L/d- ratios the power consumption would still probably be great. This might, nevertheless, be economically feasible. Although the initial cost of the equipment would be high if great air capacity is needed, the full power output would be required for only the longer waves, which occur a relatively small part of the time.

REMARKS ON MODEL TESTS

The conclusions drawn from the results of model tests (summarized in Fig. 1) depend on the assumption, made generally by investigators, that the percentage reduction in wave height obtained in the model, for a given L/d- ratio, can also be obtained in the prototype when the L/d- ratio is the same. The power required to effect such a result in the prototype is in doubt.

It has been generally assumed that the Froude law is the criterion of similarity between model and prototype; that is, it is assumed that dynamic similarity depends on gravity and inertial forces only. Laurie questioned the neglecting of viscous forces in his analysis of the experiments of the Franzius Institute,[33] pointing out how the experimenters wrote that the volumes of air required at full scale were actually 500 to 750 times less than the model would indicate by the Froude law. Certainly viscosity is likely to be a significant factor. The theoretical explanations of the pneumatic breakwater's dissipation of a wave's energy include dissipation by the surface current, by the turbulent viscosity, and by vortices. Losses of energy from all of these phenomena are dependent on viscosity. Moreover, the surface tension of water in contact with air may be significant, because it affects the formation of bubbles.

Several other investigators have found "scale effect," that is failure of results in model and prototype, to follow the Froude law. The tests by the Transportation Corps at the Beach Erosion Board in the large tank, 15 ft deep, considering it to be the prototype, permitted comparisons with tests made in smaller tanks, considering them to be the models. It appeared that

the power requirements for steep waves in deep water would be about half of what one would predict from the small laboratory tank studies. Also the disproportion appeared to increase rapidly as the L/d- ratio increased. However, the power requirements were still quite large for long waves in shallow water.

All model tests known to the writer have used the jet method of releasing the air.

Further study is being given (as of 1961) to the feasibility of designing model experiments which will account for other forces, as well as gravity and inertial forces.

CONCLUSIONS

1. The pneumatic breakwater has been demonstrated by model tests, with some verification from full-scale tests, to effect considerable attenuations (that is, percentage reductions) in the heights of waves for L/d- ratios up to 4, and possibly up to 5, when there is adequate air supply (Table 4).
2. The maximum attenuation that can be obtained with unlimited air supply declines as the L/d- ratio increases (Table 4).
3. Pneumatic breakwater equipment in its present (1960) state of development cannot furnish reasonable protection to conventional pipeline dredging equipment in areas within 1 mile of the New Jersey shore. The criterion of reasonable protection is the reduction to 3 ft or less of essentially all waves which threaten the equipment for extended periods.
4. The criterion for reasonable protection might be satisfied if pneumatic breakwater equipment could be so improved that it would be able to effect large reductions in wave heights (say 40%) for L/d- ratios up to 10. Then, two or three breakwaters in succession, spaced sufficiently far apart to act independently, could reasonably be expected to satisfy the criterion during periods of moderate sea.
5. The British equipment has a distinctive intermittent air distributor that may make it more effective than other equipment at depths near the bottom. No data are available by which the performance of the British equipment can be correlated with L/d- ratios. In accordance with wave theory a breakwater which is effective at essentially all depths should produce materially greater attenuations for L/d- ratios greater than 2 than do breakwaters that concentrate their effect in the upper layers.

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undertaking. Without the cooperation of the many individuals and organizations who supplied information the study could not have been made.

The British equipment described herein is patented by Pneumatic Breakwaters, Ltd., London, England. Pneumatic Breakwaters, Ltd., conducted the 1952 and 1956 through 1960 tests at Dover, England.

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GROINS ON THE SHORES OF THE GREAT LAKES

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SYNOPSIS

Groins are important structures in the protection and improvement of the shore. However, they are not a cure-all for shore erosion problems, as faulty design or misplacement of a groin can create additional problems or increase existing problems. Herein is included a summary of factual data on existing groins, of changes in lake levels, and other processes (the understanding of which is requisite to design), notes on design of groins for the Great Lakes, and some indication of research and costs.

INTRODUCTION

The Great Lakes involve a great expanse and volume. The shores of these lakes are subjected to wave action and other forces and processes similar to that of the oceans. Fig. 1 shows the plan of the lakes. Because these characteristics of the lakes, particularly the changes in level, are of such significance in the design of groins, they are described herein.

The Great Lakes—St. Lawrence River drainage system extends from the Gulf of St. Lawrence to the headwaters of the St. Louis River in northern Minnesota, a distance of some 2,000 miles or almost half-way across the North American continent. The entire drainage basin has an area of approximately 325,000 sq miles, of which nearly one-third is water surface. The five

Note.—Discussion open until October 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 2, May, 1961.

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Great Lakes constitute one-third of all the fresh water area in the world and cover an area nearly twice the size of the state of Illinois.

Lake Superior is the largest of the lakes having an area of 31,820 sq miles (including St. Marys River above the falls), of which 20,710 sq miles are in the United States. It is about 380 miles long, 160 miles wide at about mid-length, and has a maximum recorded depth of 1,302 ft. The United States coast line of Lake Superior, including islands, is approximately 1,427 miles. The greater part of the lake has a depth of 240 ft or more. Few places in this lake have a depth of less than 24 ft beyond one-half mile from shore. Compared with the other Great Lakes, Lake Superior is more irregular in outline

FIG. 1.—GENERAL MAP

and has deeper water, more fog, more ice, and less rain. The lake is characterized by deep water and by high rocky shores along a large part of its coast. Beaches are mainly of the pocket type in the general rock ledge shore.

Lake Michigan, the third largest of the Great Lakes, is the only one wholly within the United States. It has an area of 22,400 sq miles, is about 320 miles long, 85 miles wide in the vicinity of Milwaukee, Wis., and has a maximum recorded depth of 923 ft. It has a total length of coast line of about 1,661 miles, including islands. Approximately 25% of the Lake Michigan shore, the north and northwest portion, is composed of bedrock. The remaining 75% of the shore is composed of erodible, unconsolidated deposits. Along the west side of the lake, south of the bedrock portion and south and east of Chicago,

Ill., the shore is principally glacial till with narrow sand or sand and gravel beaches. Area along the south shore of Chicago has been reclaimed from the Lake by earth fill. From about Michigan City, Ind., at the Indiana-Michigan state line, there is an area of extensive dunes, and thereon north along the east side of the lake the shores are of sand. Along the southerly 130 miles of this sandy reach, there are high sand bluffs ranging from about 20 ft to 100 ft in height. North of this reach the bluffs become lower in elevation, and north from about Ludington, Mich., the bluffs are composed of glacial till with narrow sand beaches.

Lake Huron, second largest of the Great Lakes, has a total area of 23,000 sq miles, of which 9,100 sq miles are within the United States. It is about 250 miles long, 98 miles wide at a point above Saginaw Bay, and has a maximum recorded depth of 750 ft. Its coast line within the United States (including Saginaw Bay and the islands) is about 769 miles in length. The Lake Huron shore is generally composed of glacial till banks with narrow sand beaches.

Lake Erie, fourth largest of the Great Lakes, has a total area of 9,940 sq miles of which 4,990 are within the United States. It is about 230 miles long and near Cleveland, Ohio, it is about 57 miles wide. It is comparatively shallow, the maximum recorded depth being 210 ft and average depth about 90 ft. The easterly half of the south shore of Lake Erie is characterized by very narrow sandy beaches extending along a base of glacial till or shale bluffs, or both, that rise nearly vertically from 50 ft to 100 ft above lake level. Westerly the beaches are composed chiefly of shingle and coarse gravel and are narrow except at obstructions and at outwash plains. These beaches are backed by glacial till bluffs ranging from 50 ft to 100 ft high in various stages of erosion. The two reaches described previously are separated by Presque Isle Peninsula, a recurved sand spit extending in a northeasterly direction about 6 1/4 miles and broadens from about 250 ft in width at the neck to about 1 1/4 miles toward the distal end.

Lake Ontario, the smallest in area of the lakes, although it has a larger volume than Lake Erie, has a total area of 7,540 sq miles, of which 3,560 sq miles are within the United States. It is about 180 miles long and about 50 miles wide near Rochester, N. Y. Lake Ontario is comparatively deep, its maximum recorded depth being 778 ft. Its coast line within the United States is about 331 miles in length including the islands. The south shore of the lake east to Mexico Bay, just north of Oswego, N. Y., is quite regular with shallow water extending about one-half mile from shore. East of Mexico Bay are several shallow ponds practically cut off from the lake by narrow ridges of shore. Numerous creeks empty into these ponds. In the easterly end of the lake, the coast north of Stony Point is characterized by numerous islands and bays. In general, its shore consists of glacial till banks with narrow beaches.

The principal flow is through the lakes themselves and their connecting channels. Water flows from Lake Superior through the St. Marys River, dropping about 20 ft, into Lake Huron. Lakes Huron and Michigan are connected by the Straits of Mackinac, a waterway of sufficient size to allow the two lakes to maintain identical levels and in fact react hydraulically as a single lake. From Lake Huron the water surface drops about 8 ft to Lake Erie, flowing through the St. Clair River, Lake St. Clair, and the Detroit River. The water surface drops about 326 ft between Lake Erie and Lake Ontario, the flow being over Niagara Falls and through the Niagara River. The flow from

Lake Ontario is through the St. Lawrence River (the improved St. Lawrence Seaway) to the Gulf of St. Lawrence.

LAKE LEVELS

The Great Lakes have stood at their present elevations for several thousand years. Recent geological studies suggest that Lake Michigan and Lake Huron reached their present elevations less than two thousand years ago. In the past, water levels in all the lakes have been both higher and lower by considerable amounts. The old higher level stages are indicated by beach deposits, wave cut cliffs and other features which are now abandoned. The lower stages are indicated by features such as sand and shell zones in deep water clays and by deep valleys in the lower courses of rivers, now partially or wholly filled by fine-grained sediment.

There are three definite types of fluctuations in the present levels of the lakes. These may be described as (1) long term variations, caused principally by long term trends in precipitation; (2) seasonal variation caused by seasonal distribution of rainfall and the freezing and thawing schedule of the year; and, (3) temporary or short term fluctuations caused mainly by wind setup, and rapid barometric pressure changes.

The levels of Lake Superior and Lake Ontario are controlled by man-made structures. The control of Lake Superior is effected by compensating works in the St. Marys River, reducing the range of fluctuation by reducing both highs and lows, but allowing the lake surface to fluctuate essentially within the natural range. The control is governed by treaty between the United States and Canada through the International Joint Commission; the agreement being that the levels would be held "as nearly as may be" between the elevations of 602.1 and 603.6 above mean tide at New York, N. Y. The existing works were completed in 1921. Lake Ontario is controlled "as nearly as may be" between the elevations of 244 and 248, except under conditions as previously agreed upon, by regulating works at the Iroquois Dam. These works were completed in July 1958 and are operated together with the production of hydroelectric power. Regulation of the remaining lakes is now under study by the U. S. Army Corps of Engineers, North Central Division in Chicago, Ill. This regulation would entail reducing the seasonal highs and lows of these lakes without interfering with the agreed-upon regulation of Lakes Superior and Ontario. During the period of record 1860-1958, long term high levels have occurred on the four uncontrolled lakes at intervals ranging from 5 yr to 29 yr. The occurrences of long term fluctuations are shown in Table 1.

The occurrence of long term high or low water fluctuations are without doubt related to precipitation. However, students of the lakes have been unable to determine a constant lag or a definite formula relating precipitation to lake levels. A comparison of extreme high and low monthly mean levels are given in Table 2.

Seasonal fluctuations are superimposed upon the long term levels and are the only type that occur in a relatively regular pattern. The low point of the seasonal variation generally occurs in late winter before spring rains and prior to ice and snow thaws. The high occurs during the summer months following the thaw. Fig. 2 is a series of hydrographs of levels of each lake during the period 1926-1958. Table 3 gives the average time of occurrences of the highs and lows and the average range of seasonal fluctuation on each lake.

Short term or temporary fluctuations generally range in duration from a few minutes to several hours, depending upon the cause and the characteristics of this cause. These fluctuations are, of course, superimposed upon the still water level existing at the time, that would include both the prevailing long term and seasonal increment of level. The temporary fluctuation is, in fact, an unbalancing or tilting of the water surface within the lake. Table 4 gives, for specific gage locations on the various lakes, the maximum heights of

TABLE 1.—OCCURRENCE OF HIGH LAKE LEVELS

Lake Michigan-Huron		Lake Erie		Lake Ontario	
Year	Interval, in years	Year	Interval, in years	Year	Interval, in years
1861		1862		1870	
	15		14		6
1876		1876		1876	
	10		8		10
1886		1884		1886	
	22		29		22
1908		1913		1908	
	10		16		21
1918		1929		1929	
	11		18		18
1929		1947		1947	
	23		5		5
1952		1952		1952	

TABLE 2.—COMPARISON OF HIGH AND LOW LAKE LEVELS

Lake	Average Level	Highest One-Month Average		Lowest One-Month Average		Range, in Feet
		Level	Month and Year	Level	Month and Year	
Superior	602.2	604.05	Aug. 1876	599.98	April 1926	4.07
Michigan-Huron	580.6	583.68	June 1886	577.73	Feb. 1926	6.33
Erie	572.3	574.70	May 1952	569.43	Feb. 1936	5.27
Ontario	246.0	249.29	June 1952	242.68	Nov. 1934	6.61

Levels are elevation in feet above MSL at New York, N. Y.

short term fluctuations occurring at average frequencies of once each in 1 yr, 2 yr, 5 yr and 10 yr.

ICE EFFECT ON THE SHORE

Because the Great Lakes are composed wholly of fresh water, there is considerable ice formation. These ice conditions may be beneficial or harmful in their effect on the shore or shore structures. The harmful effects result principally from the battering of structures by floating ice, particularly during

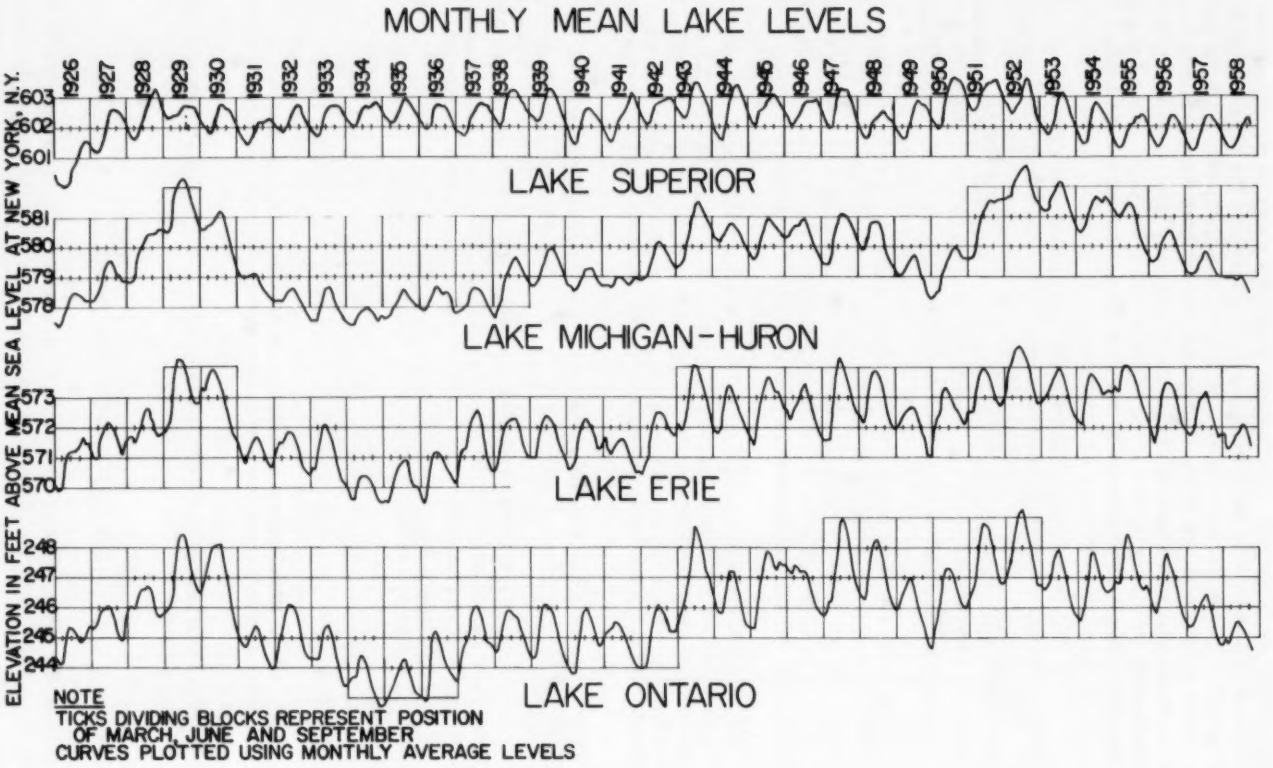

FIG. 2.—HYDROGRAPHS OF THE LAKES

TABLE 3.—OCCURRENCE OF SEASONAL FLUCTUATIONS

Lake	Usual Month of Occurrence		Average Range, in Feet
	High	Low	
Superior	September	March	1.2
Michigan-Huron	July	February	1.1
Erie	June	February	1.6
Ontario	June	January	1.8

TABLE 4.—SHORT PERIOD FLUCTUATIONS

Lake	Gage Location	Period of Record	Maximum Height of Fluctuation at Various Frequencies of Occurrence[a]			
			1 yr	2 yr	5 yr	10 yr
Superior	Two Harbors, Minn.	1941-1952	1.4	1.6	1.8	2.1
	Marquette, Mich.	1902-1952	1.4	1.6	1.8	2.0
	Pt. Iroquois, Mich.	1930-1952	1.3	1.5	1.9	2.5
Michigan	Mackinaw City, Mich.	1899-1952	1.0	1.1	1.4	1.6
	Ludington, Mich.	1946-1952	1.0	1.1	1.2	-
	Calumet Hbr. Mich.	1922-1952	1.8	2.1	2.4	2.6
	Milwaukee, Wis.	1903-1952	1.3	1.4	1.7	1.8
	Sturgeon Bay Canal, Wisc.	1945-1952	1.3	1.6	1.7	-
Huron	Harbor Beach, Mich.	1960-1952	1.0	1.1	1.2	1.4
	Fort Gratiot, Mich.	1937-1950	1.7	1.9	2.1	2.5
	Gibraltar, Mich.	1941-1952	2.1	2.2	2.5	2.7
Erie	Toledo, Ohio	1905-1952	3.4	3.8	4.2	4.5
	Put-in-Bay, Ohio	1941-1952	2.0	2.2	2.3	2.4
	Cleveland, Ohio	1940-1952	1.5	1.7	1.9	2.0
	Buffalo, N. Y.	1902-1952	5.0	5.5	6.5	7.5
Ontario	Oswego, N. Y.	1902-1952	1.2	1.4	1.7	2.2
	Tibbetts Pt., N. Y.	1899-1951	1.7	1.8	2.3	2.7

[a] Heights are in feet.

heavy ice floes following spring breakup, and from uplift pressures on structures. Beneficial effects result when the shore and shore structures are protected from wave action by the following occurrences: (1) Usually in early winter wave swash and spray forms ice on shore and shore structures, at times the ice builds to a height of 12 ft to 15 ft; (2) in midwinter ice forms a solid covering extending as much as several miles offshore; and (3) storms often break up the ice fields in early winter and build windrows offshore that act as breakwaters. These windrows at times attain a height of 10 ft to 15 ft.

WAVES

As mentioned previously, waves experienced on the Great Lakes are of similar dimensions to those of the ocean coasts. The most extensive recent published works on waves on the Great Lakes is a series of Technical Memoranda of the Beach Erosion Board, prepared by Thorndike Saville, Jr., M. ASCE, in 1953.[2,3,4] These are the results of a study made by the Beach Erosion Board staff and including hindcasts from United States Weather Bureau (USWB) 6-hr synoptic charts for the 3 yr period 1948-1950, inclusive. The wave characteristics were obtained from curves derived by Sverdrup and Munk, as revised by Arthur and later by Charles Bretschneider, M. ASCE. Additional hindcasts were made in the North Central Division, Corps of Engineers, and the Milwaukee, Chicago and Buffalo Districts thereof in 1953, under the general supervision of the writer, using the 5-yr period of record 1948-1952, inclusive.[5] In this study fetches used were shore-to-shore straight line distances in the direction of the wind, as determined at the shore observation stations, and wave heights were determined by the same hindcast method used by the staff of the Beach Erosion Board. Wave heights occurring with an average frequency of one each year and once in ten years are given in Table 5.

LITTORAL DRIFT

Wave generated currents are a principal factor in beach stability or equilibrium. Material is moved and redistributed along the beach by waves and wave generated currents as bed load and as suspended load, when such material is available. The predominant direction of the currents depends on the predominant angle of approach of the waves, which of course is in turn dependent upon prevailing wind direction. Predominant direction of current is not necessarily the predominant direction of littoral drift, because there may be a littoral barrier or absence of source of material to the updrift. The determination of predominant littoral drift by computation of resultant wave energy, consideration of material source, and use of indications, such as

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2 "Wave and Lake Level Statistics for Lake Michigan," by Thorndike Saville, Jr., Beach Erosion Bd., Tech. Memorandum No. 36, 1953.

3 "Wave and Lake Level Statistics for Lake Erie," by Thorndike Saville, Jr., Beach Erosion Bd., Tech. Memorandum No. 37, 1953.

4 "Wave and Lake Level Statistics for Lake Ontario," by Thorndike Saville, Jr., Beach Erosion Bd., Tech. Memorandum No. 38, 1953.

5 "Additional Wave Statistics for Stations on Lake Michigan and Lake Erie," by Charles E. Lee, Annual Bulletin, Beach Erosion Bd., 1958.

accretion at barriers, permits the following conclusions:

1. Lake Superior.—The drift along the Minnesota shore of the lake generally is from west to east northeasterly of Grand Marais and from east to west for the area westerly to Duluth, Minn. Along the southern shore from Duluth to the peninsula westerly of Marquette, Mich. the drift trends from east to west, then reverses and flows generally west to east to about midway the distance to the St. Marys River then becomes quite strongly from west to east.

TABLE 5.—HINDCAST WAVE HEIGHTS—ONE YEAR AND TEN YEAR FREQUENCY

Lake	Station	Wave Ht. in Feet[b]		Source[d]
		1-yr Frequency	10-yr Frequency	
Superior	Brule River, Minn.	20	--	—[c]
	Carvers Bay	27	-	—[c]
	Little Lake, Mich.	22	-	—[c]
Michigan	Baileys Hbr., Wis.	9.5	13.0	Saville[2]
	Milwaukee, Wis.	12.0	17.0	Lee[5]
	Chicago, Ill.	5	12.5	Saville[2]
	Muskegon, Mich.	14	22.5	Lee[5]
	Frankfort, Mich.	17.5	23.5[a]	Saville[2]
Huron	North Point, Mich.	9	-	—[c]
	Harbor Beach, Mich.	13	-	—[c]
	Port Huron, Mich.	8	-	—[c]
Erie	Monroe, Mich.	9.0	13.0	Saville[3]
	Cleveland, Ohio	9.5	12.5	Lee[5]
	Erie, Pa.	11.0	14.0	Saville[3]
	Buffalo, N. Y.	15	20.5	Lee[5]
Ontario	Hamilton, Ontario	11.5	27.0[a]	Saville[4]
	Olcott, N. Y.	9	-	—[c]
	Rochester, N. Y.	11.5	16.0	Saville[4]
	Oswego, N. Y.	11	-	—[c]
	Stoney Pt., N. Y.	15.0	17.5	Saville[4]

[a] Extrapolation of curves beyond presented limits.
[b] Deep water wave based on full year (12 months).
[c] From "Preliminary Examination Report on Property Damage on the Great Lakes Resulting from Changes in Lake Levels," by Great Lakes Div., Corps of Engrs. (now U. S. Army Engr. Div., North Central), Chicago, Ill., 1952.
[d] Superscript numbers refer to footnotes used in text.

2. Lake Michigan.—Along the western shore, in the vicinity of the north half of Door County Peninsula, the direction of drift varies. The upcoast and downcoast direction components of wave energy are practically equal. North of Two Rivers, the drift is predominantly northward, and to the south, the drift is predominantly southward. In the vicinity of Milwaukee, Wis., the drift has a much stronger component toward the south. The over-all tendency to southward drift continues to some point below Chicago, where the directional trend of the coastline causes a reversal of drift. Along the eastern shore, between this nodal zone and about Ludington, the predominant drift is to the

south. From near Ludington northward, the drift varies, but is predominantly northward.

3. Lake Huron.—In the northerly half of the shore, between Saginaw Bay and the Straits of Mackinac, the drift direction is quite variable. From that midpoint southward to Saginaw Bay, the trend is southward. Saginaw Bay is considered a complete barrier to littoral drift. Just north of Harbor Beach, the drift resumes a southward predominence to Port Huron, Mich.

4. Lake Erie.—West of Cleveland, Ohio, the direction of drift varies as in certain areas the predominant direction is to the west, and in others is to the east. However, from Cleveland to Buffalo, the drift is predominantly eastward.

5. Lake Ontario.—From Hamilton, Ontario, to a short distance easterly of Niagara Falls, the drift is generally westward, then becomes easterly and, because of shoreline direction, becomes northerly from about Oswego.

EXISTING GROINS

Counts of groins were available for eleven sample areas on Lake Michigan, Erie, and Ontario. No information was obtained for areas on Lakes Superior and Huron. The information was obtained from cooperative beach erosion control studies made by the U. S. Corps of Engineers and such supplemental information as could be obtained. The sample areas are as follows, the dates in parenthesis refer to the approximate date of the count:

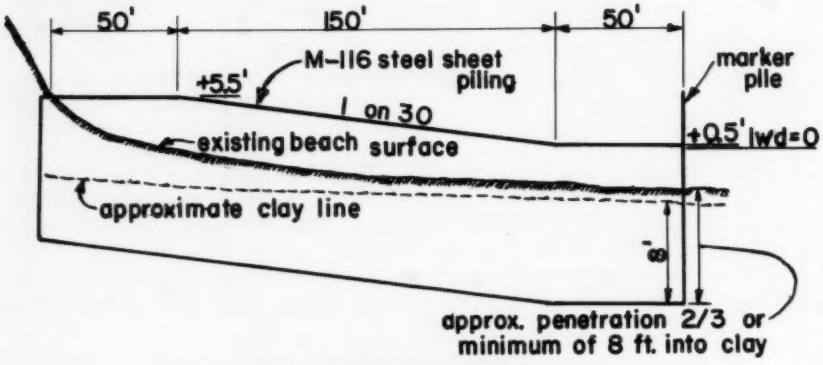
1. Racine County, Wis. (1951), along 14 miles of shore line of the west shore of Lake Michigan;
2. City of Kenosha, Wis. (1954), along 4.5 miles of shore line of the west shore of Lake Michigan;
3. State of Illinois (1949), along 58 miles of shore line of the southwest and south shore of Lake Michigan;
4. Berrien County, Mich. (1956), along 42 miles of shore line of the east shore of Lake Michigan;
5. Chagrin River to Fairport Harbor, Lake County, Ohio (1949), along 10 miles of shore line of the southeast shore of Lake Erie;
6. Sandusky to Vermilion, Ohio (1951), along 20 miles of shore line of the southeast shore of Lake Erie;
7. Presque Isle Peninsula, Penna. (1958), along 6.25 miles of shore line, near Erie, of the southeast shore of Lake Erie;
8. Hamlin Beach State Park, N. Y. (1954), along 12,000 feet of shore line, west of Rochester, of the south shore of Lake Ontario;
9. Braddock Bay State Park, N. Y. (1954), along one mile of shore line, west of Rochester, of the south shore of Lake Ontario;
10. Fair Haven State Park, N.Y. (1954), along 2 miles of shore line, southwest of Oswego, of the south shore of Lake Ontario;
11. Selkirk Shores State Park, N. Y. (1953), along 6,000 ft of shore line, northeast of Oswego, of the east shore of Lake Ontario.

These data are tabulated in Table 6 according to permeability and material of construction. Of the 841 groins examined, there were 109 or 13% permeable and 732 or 87% impermeable. Of the permeable groins, 55 of these, or 50%, are of precast concrete construction, most of which are the patented type illustrated in Fig. 3. In all there are 209 groins of concrete construction, making up 25% of the total. Of this 209, there are 59 groins or 28% permeable.

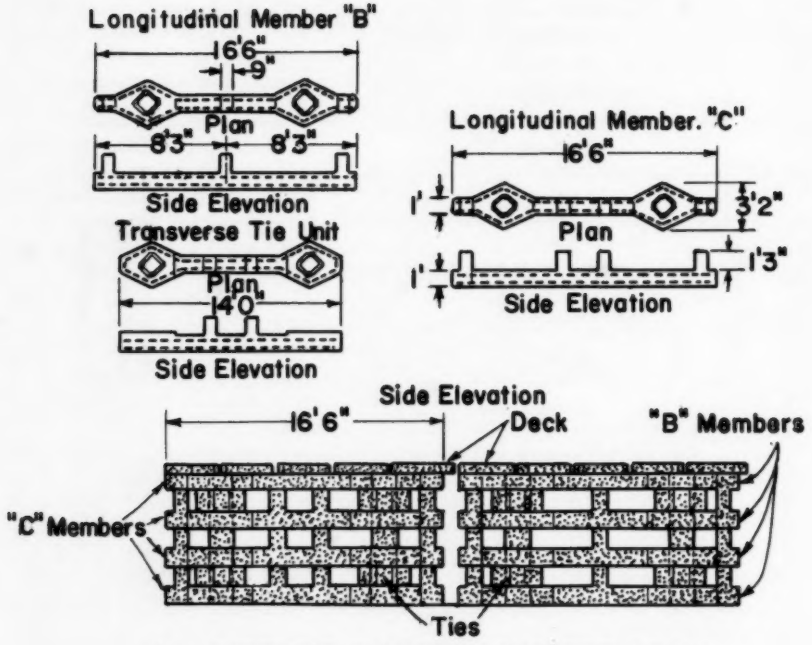
TABLE 6.—TYPES OF GROINS IN SAMPLE AREAS

Location	Impermeable													Permeable								
	Timber Crib (3)	Steel Sheet Pile (4)	Steel Sheet Pile (F)	Steel Sheet Pile (T)	Timber	Concrete	Concrete (T)	Concrete Block	Cellular Steel	Stone	Steel Crib	Concrete Pile	Sub-total	Concrete Block	Precast Concrete	Timber	Steel Sheet Pile	Timber Crib	Dumped Riprap	Boulder	Sub-total	Total
Racine Co., Wis.	4				22	6		4		21	2(6)		59		17						17	76
Kenosha, Wis.							1	12				7	20	4							4	24
State of Illinois	66	66	2		60	9		1		1			205		36	1	10				47	252
Berrien Co., Mich.		59		5	38	3			7				112			3					3	115
Lake Co., Ohio	7	2				47(5)			2	43			101		1	4					5	106
S-V, Ohio (1)	99	8			12	38	6	5		16			184			7		8	6	10	31	215
Presque Isle, Pa.	4	19				5	5		2	2			37		1						1	38
Hamlin Beach (2)						1				6			7								0	7
Braddock Bay (2)										4			4								0	4
Fairhaven (2)	1												1							1	1	2
Selkirk (2)										1	1		2								0	2
Total	181	154	2	5	132	109	12	22	11	94	3	7	732	4	55	15	10	8	6	11	109	841

(1) Sandusky to Vermilion, Ohio
(2) New York State Beaches
(3) Generally stone filled, some sand filled, some with concrete cap
(4) Some with timber piles, few are partially timber sheeting
(5) Some short stub groins attached to seawall
(6) Steel oil drums and damaged boilers, stone or sand filled
(F) Groins F-shaped in plan
(T) Groins T-shaped in plan



(a) STEEL SHEET PILING GROIN
(Fort Sheridan, Ill.)



(b) PATENTED PRECAST CONCRETE GROIN
(S.M. Wood)

FIG. 3.—TYPES OF GROINS

The remainder of the groins are distributed as follows: 189 or 22% timber crib; 182 or 22% steel sheet pile, of cantilever, cellular or crib construction; 147 or 17.5% timber pile and sheeting; and 111 or 13% placed stone, dumped riprap or boulders.

Table 7 lists 831 groins from the 11 sample areas according to length, dimensions were not available for 10 groins along the state of Illinois shore. These statistics show 531, or 64%, are 100 ft or less in length, 174, or 21%, are 101 to 150 ft in length and the remainder range to over 300 ft in length. This predominance of short groins is due largely to the large percent of private ownership, because private owners are usually unable or unwilling to make the expenditure to provide for other than minimum protection. Many of the short groins are spurs attached to bulkheads or seawalls.

Table 8 presents a list of 626 groins according to ownership. Ownership information for the area between Sandusky, Ohio, and Vermillion, Ohio, was not available, therefore, the number of groins tabulated herein is less than in Tables 6 and 7. Of these 626 groins, 318, or 51%, are privately owned, 74,

TABLE 7.—NUMBER OF GROINS ACCORDING TO LENGTH IN FEET

Location	0 - 100	101 - 150	151 - 200	201 - 250	251 - 300	Over 300	Total
Racine Co., Wis.	48	23	2	1	1	1	76
Kenosha, Wis.	8	3	6	3	2	2	24
State of Ill.	122	72	25	13	1	9	242[a]
Berrien Co., Mich.	67	35	6	4	1	2	115
Lake Co., Ohio	100	3	1	-	-	2	106
S-V, Ohio	173	32	8	1	-	1	215
Presque Isle, Pa.	9	1	0	11	12	5	38
Hamlin Beach, N.Y.	-	-	-	1	1	-	2
Selkirk, N. Y.	3	3	-	-	-	1	7
Braddock Bay, N.Y.	-	2	-	2	1	-	4
TOTAL	531	174	48	36	19	23	831

[a] Dimensions of ten groins unknown.

or 12%, are city owned, and 94, or 15%, of unknown ownership. The remaining 22% are owned by states, the United States, Racine County, Wis., Park or Sanitary Districts, cooperatively owned by states and the United States, or by unknown public agencies.

In recent years public agencies have shown greater interest in the protection of the shores of the Great Lakes, including protection by the use of groins. There is, of course, greater federal activity since federal participation in the cost of construction of protective works was authorized by Public Law 727, 79th Congress, 2nd Session (1946), and amended by Public Law 826, 84th Congress (1956). Whereas many groins have been authorized or recommended for construction with federal participation in their cost, as of May 1959, only twelve have been constructed. These groins consist of one of stone at Selkirk Shores State Park on Lake Ontario, in cooperation with the state of New York, two of steel cells, and nine of steel sheet pile (cantilever) at Presque Isle Peninsula on Lake Erie, in cooperation with the state of Pennsylvania. Those at Presque Isle were constructed concurrently with direct

placement of sand fill on the beach. The material was dredged from the shore along the landward side of the peninsula.

Each type of groin referred to in Table 6 is described in following paragraphs. Several groins of unusual construction, of which some were included under a general classification in the table, are also described. The groins listed are impermeable unless otherwise defined. Plans of two types of groins, the cantilever steel sheet pile groin and the precast concrete permeable groin, patented by Sidney M. Wood, are shown on Fig. 3.

Steel Sheet-Pile Groins.—Cantilever steel sheet-pile groins are described first because this type of groin has proved most economical in many areas along the lakes. They consist of interlocked steel sheet pile, generally of material weighing 22 psf to 32 psf, driven along a straight center-line, generally perpendicular to the shore. When there is a layer of lacustrine material in the foundation, penetration of at least two times the exposed portion is often

TABLE 8.—NUMBER OF GROINS ACCORDING TO OWNERSHIP

Location	Private	City	State	U. S.	County	District[a]	U. S. and State	Public[b]	Unknown	Total
Racine Co., Wis.	42	9	–	1	24	–	–	–	–	76
Kenosha, Wis.	1	23	–	–	–	–	–	–	–	24
State of Illinois	143	42	16	19	–	26	–	–	6	252
Berrien Co., Mich.	20	–	–	–	–	–	–	7	88	115
Lake Co., Ohio	106	–	–	–	–	–	–	–	–	106
Presque Isle, Pa.	6	–	20	4	–	–	8	–	–	38
Hamlin Beach, N.Y.	–	–	7	–	–	–	–	–	–	7
Braddock Bay, N.Y.	–	–	4	–	–	–	–	–	–	4
Fair Haven, N.Y.	–	–	2	–	–	–	–	–	–	2
Selkirk, N.Y.	–	–	1	–	–	–	1	–	–	2
TOTAL	318	74	50	24	24	26	9	7	94	626

[a] Park or Sanitary Districts
[b] Actual public agency unknown

required. This penetration is reduced when there is a firm foundation material. It may be constructed with or without wales. Often the outer end, which is most susceptable to damage, is reinforced to withstand ice pressures. Cantilever sheet-pile groins are found extensively along the state of Illinois' shore,[6,7] Berrien County, Mich., shore and at Presque Isle Peninsula, Erie, Penna.[8,9] The Corps of Engineers constructed ten groins of this type at Fort Sheridan, Ill., and designed nine others constructed at Presque Isle Peninsula,

[6] "Cooperative Beach Erosion Control Study of Illinois Shore of Lake Michigan," House Document No. 28, 83rd Congress, 1st Session.

[7] "Lake Michigan Erosion Studies," by John R. Hardin and William H. Booth, Jr., Transactions, ASCE, Vol. 118, 1953, pp. 39-50; and Discussion by Charles E. Lee, pp. 55-59.

[8] "Cooperative Beach Erosion Control Study of Presque Isle Peninsula, Erie, Pa.," House Document No. 231, 83rd Congress, 1st Session.

[9] "Case History of Shore Protection at Presque Isle Peninsula, Pa.," by Charles E. Lee, Proceedings, Third Conf. on Coastal Engrg., Council on Wave Research, 1952.

in cooperation with the state of New York. When last inspected the groins at Presque Isle were functioning as expected by maintaining a protective fillet on the updrift side. The Commonwealth has a much needed project underway to provide periodic nourishment to the groin field.

In addition to the straight steel groins constructed perpendicular to the shore, there are also instances in which groins are constructed at various angles to the shore, others form a "T" or an "F." Those constructed at an angle with the shore generally are based on the premise that the acute angle (considering the predominant direction of drift) will collect and retain a larger percentage of available drift, and that those forming an obtuse angle will retain only a portion of the drift and allow the passage of the remainder, thereby achieving the effect of permeability, nourishing the downdrift side of the groin to some extent and decreasing the scour potential. From observation, it is concluded that groins forming an obtuse angle with the shore are generally ineffective and that the effect created by those forming an acute angle can be duplicated at less cost by a straight perpendicular groin extending a like distance offshore. The "F" and "T" groins were obviously designed on the premise that scour potential would be decreased by diffracting the impinging waves. There are conflicting opinions on the value of this type of construction. Those observed have been effective. However, in all cases, the groins were relatively short and located in areas of ample drift with some variation in direction, making it reasonable to assume that straight groins without spurs might have similar effectiveness.

Steel Sheet-Pile Cellular Groins.—Steel sheet-pile cellular groins are constructed of interlocking steel sheet pile forming connecting cells of circular or elliptical shape and filled with sand or stone. Little penetration of the piles is needed because the cells are generally filled immediately after construction. This type of groin is not in general use. However, it is a valuable design in cases in which alternative materials are expensive and penetration cannot be obtained, or when ice or wave action is severe.

Timber Crib Groins.—Timber crib groins consist of a series of connected, stone filled conventional cribs made of rectangular or notched round timbers secured to timber piles at the four corners. They are used in many areas throughout the lakes, especially where the groins were constructed in earlier years when timber was more plentiful and less expensive.

Timber Pile and Sheeting Groins.—Timber pile and sheeting groins are many variations of the timber groin. The most extensively and successfully used type consists of two or more rows of vertical driven sheeting with wales on both sides along the top and about midway between the top and the beach surface, and round piles driven at intervals along the entire length. Penetration of the piles and sheeting is dependent on foundation conditions, but the sheeting should extend below the depth of maximum expected scour.

Concrete Groins.—The two types of impermeable concrete groins used most frequently on the Great Lakes are monolithic construction and concrete block construction. The blocks are of various sizes and shapes. In addition, there are a few units constructed of concrete sheet piles tied together by channel wales.

A rather unusual type of groin, constructed of concrete pipe, is located at South Shore Park, Milwaukee, Wis.[10] However, variations of this design are found at other locations. Nearshore breakwaters of sand filled concrete pipe

10 "Concrete Shore Protection," by Portland Cement Assoc., Chicago, Ill., 1955.

are located at South Haven, Michigan, on the east side of Lake Michigan and at Fairport, Ohio on Lake Erie.

The groins at South Shore Park were constructed of sections of 32-in., 42-in., and 60-in. concrete pipe (progressively increasing in size seaward from the shore end) placed adjacent to each other, filled with gravel, and capped with concrete. The sections of pipe are tied together by channel wales on each side secured by tierods. The groins were constructed without concurrent placement of sand fill in an area where there is little littoral drift. Therefore, the groins have not caused accretion of significant dimensions, but it is believed that the rate of erosion in the area has decreased.

Stone Groins.—Stone groins are used more often on Lake Erie and Lake Ontario than on the other lakes. This is probably due to the high cost of stone on the southerly portion of Lake Michigan; Lake Superior is not severely troubled by erosion problems and Lake Huron is less heavily developed than the others. The stone groins are generally constructed of a trapazoidal section with a top width of about 3 ft to 5 ft. Common construction practice is to place the core material to about half the design height, run the placing equipment to the outer end, then work shoreward placing the slope and crest stone to finished height. Other stone groins commonly used are of dumped riprap or boulders, both permeable.

Permeable Groins.—There are various types of permeable groins used. The precast concrete type patented by Wood is used rather widely. These consist of precast reinforced concrete units shaped like flat dumbbells. The units are threaded on concrete piles to form cribs. The permeability is varied by varying the number of longitudinal members. The lengths of units range from 6 ft to 14 ft with corresponding weights of 0.75 tons to 1.5 tons. The width of the groin is approximately equal to length of the unit. A concrete slab deck is often placed on the top of groins to form a usable top surface. See Fig. 3 for additional details.

Although groins of the preceding design are found at many locations, the writer is most familiar with those on the shore fronting Northwestern Univ. at Evanston, Ill. These groins have required little maintenance considering their length of existence, but they have not eliminated the damage to the shore.

Other commonly used types of permeable groins are constructed of concrete block, steel sheet pile, and timber. Steel sheet-pile type are usually slotted with increasing permeability toward the outer end; the timber type are generally constructed of vertical sheeting with space left between sheets.

DESIGN CONSIDERATIONS

No strict groin design can be established that would be applicable to all areas. The use of groins should be recommended only after a thorough study of their applicability in the specific area, their effect on adjacent shore areas, the need for the concurrent placement of sand fill, and the economics of periodic nourishment of the beach without groins. The material, type, dimensions, spacing and cross section of each individual groin must be determined on the basis of intended purpose, location, exposure, drift characteristics, availability of materials, and limits on construction methods. In general, it is considered that impermeable groins should be used on the Great Lakes. General design criteria and methods follow.

First step in design of groins is the determination of the beach dimensions required for protection of the area or to service the expected beach attendance. From an economy standpoint, it is usually not possible to provide full protection from the most severe conditions that may occur, but only against those having a reasonable frequency of occurrence. In this respect, the elevation of the maximum monthly lake level, that is expected to occur once in 20 yr, together with the height of the maximum temporary fluctuations expected to occur once each year superimposed upon it is generally a reasonable still water design level. The design elevation of the beach berm would be a distance above this design lake level equal to the run-up of the maximum wave expected to approach without breaking. This design berm elevation would vary according to location, as both temporary fluctuation and impinging wave height depend principally on fetch and nearshore hydrography. Generally design berm elevations fall within the following ranges above mean tides at New York.

Lake Superior, 609 ft to 611 ft
Lake Michigan - Huron, 588.5 ft to 590.5 ft
Lake Erie, 580.5 ft to 582.5 ft
Lake Ontario, 253 ft to 255 ft

The horizontal shore section of groins should be securely tied to high ground at the shore end to prevent flanking and ordinarily have a top elevation not lower than the general height of berms of existing beaches and a length equal to the berm width of the anticipated beach. Barrier groins, that are intended to completely block passage of littoral drift, or to reduce it considerably, should be higher than the anticipated beach berm. Also, in the case of stone groins, it is desirable to have the top elevation about 1 ft higher than the anticipated beach berm elevation to compensate for the spaces between cover stones. The intermediate sloped section should not be steeper than the slope of the existing foreshore, and should approximate the slope of the anticipated beach. For construction ease, the top elevation of the seaward section is governed by the expected still water elevation at the time of construction. For stone construction, the minimum height and crest width of groins is determined by the size stone needed to withstand wave forces. Groin length and spacing are inter-related and are governed by (1) the angle the beach normally makes with the generalized shoreline and (2) the minimum width of beach required on the downdrift side of the groins. The beach alinement is, of course, a function of direction of wave attack. Penetration of cover stone is determined from depth of scour that may be expected to occur during critical conditions. Blankets of spalls or crushed stone are used under stone groins or jetties to minimize settlement due to scour. More detailed information on design of groins is available.[11]

Analysis for structural stability depends on the construction material to be used. For a rubble mound structure on a sand beach, stability is usually attained by determining the size of individual stones, together with the placement slope, necessary to withstand the force of the design wave. A method of determining the design wave, which is usually related to the depth of water at the toe of the structure, is also adequately covered in a previous reference.

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11 "Shore Protection - Planning and Design, Technical Report No. 4," by Beach Erosion Bd., Corps of Engrs., Revised 1957.

The recommended guide for determination of stone size and slope is the expression[12]

$$W_r = \frac{W_{sr} H^3}{K_d (S_r - 1)^3 \cot a} \quad \ldots\ldots\ldots\ldots\ldots (1)$$

in which W_r is the weight of individual stones, W_{sr} defines the specific weight of rock, H denotes the wave height, K_d refers to a constant equal to 3.2 for no damage conditions, S_r is the specific gravity of rock, and a denotes the angle the breakwater slope makes with horizontal. The guide expression is based on model tests of waves approaching a structure at an angle of 90°. It should be used together with experience in the locality. In most cases side slopes should not be steeper than 1 vertical to 1.5 horizontal in order to prevent collapse of the slope itself. Foundation conditions must be given consideration, particularly for the larger groins. For poorer foundations, it may be necessary to use a flatter side slope or provide an apron of bedding material to distribute the weight over a larger area, or both.

Stability of cellular or cantilever groins is particularly dependent upon the horizontal pressure exerted by wave attack and upon the foundation material. The best of the known methods of estimating pressure exerted by waves of various characteristics has been given by the Beach Erosion Board.[11] Critical considerations for design of steel sheet pile cellular groins are interlock stress and tilting of the cells. The design method proposed by E. M. Cummings,[13] F. ASCE, using a factor of safety of 1.5 for tilt and 2.0 for interlock stress, appears satisfactory. The design of cantilever groins, including materials, penetration, wales, and other strengthening devices is adequately covered by others.[11]

FILLING PROCESSES

The Great Lakes Division and Chicago District, Corps of Engineers, made a study of rate of filling of a five groin system at Fort Sheridan, Ill., between October 1951 and October 1952. This study report, on which the design of the cantilever steel sheet pile groins was based, was published as House Document No. 28, 83rd Congress, 1st Session. Analysis of this report is available.[7]

The five groin system was constructed during May-August 1951. A study was made of the system by analysis of 6 repetitive topographic and hydrographic surveys made in October and December 1951 and April, May, August, and October 1952.[14] Over the 12 month period, the system collected 21,600 cu yd of material, an average of 1800 cu yd per month. The rate of accumulation according to time was as follows: Between October 12 and December 4, 1951, an average of 3,120 cu yd per month; between December 4, 1951 and April 18, 1952, an average of 1,020 cu yd per month; between April 18, 1952 and May 28, 1952, an average of 4,570 cu yd per month; between May 28, 1952 and August 12, 1952, an average of 210 cu yd per month; and, between August 12, 1952 and October 8, 1952, an average of 2,245 cu yd per month. The study indicated that rates of filling for the Fort Sheridan area could be correlated

12 "Design of Quarry-stone Cover Layers for Rubble Mound Breakwaters," by R. Y. Hudson, U. S. Army Engr. Experiment Sta., Research Report No. 2-2, 1958.

13 "Cellular Cofferdams and Docks," by E. M. Cummings, Proceedings, ASCE, Vol. 83, No. WW 3, September, 1957.

14 "Filling Pattern of the Fort Sheridan, Ill. Groin System," by Charles E. Lee, Proceedings, Fourth Conf. on Coastal Engrg., Council on Wave Research, 1953.

directly to an index of wave energy when corrected for direction, impounding ability of the groins, ice effect, and lake level.

Principal conclusions that may be drawn from the study are that (1) in the Fort Sheridan area storm waves are the principal motivating force of littoral drift, causing a varying rate in the filling process; (2) initial accretion occurs lakeward of the outer ends of the groins; (3) initial filling does not necessarily occur at the updrift groin, but rather the groin may act as an energy dissipator; and (4) the storm waves cause irregular filling patterns whereas the less steep (steepness = H/L, in which L is the wave length) waves smooth the profile.

COSTS

Little information on costs have been obtained. However, costs of specific structures on Lakes Michigan, Erie, and Ontario are given. There are many factors exclusive of cost of materials and labor that affect groin cost. These factors include the characteristics of the nearshore and foreshore that affects the quantity of material per foot of groin; location and accessibility of the site affects mobilization of construction equipment as well as transportation cost of materials; characteristics of topography and foundation as well as construction methods required affects the ease and rapidity at which the construction may be accomplished; and the availability of nearby natural construction materials affects the transportation cost.

The following information was obtained for groins constructed on Lake Michigan:

1. Ten steel sheet pile cantilever groins were constructed in 1951 and 1953 at Fort Sheridan, Ill. The total length of the ten groins was 2,347 linear ft. The cost of the groins was $115,701 or $49 per foot of groin.
2. In 1949 two groins were constructed at Rogers Park in Chicago, Ill., of timber piles with triple Wakefield sheeting. The cost of the 240 ft total length of groins was $27,100 or $113 per foot of groin.

In 1951 the state of Ohio constructed four test groins on state owned property at Camp Perry in Ottawa County. The cost of the four groins was $17,490.88. The groins are described as follows:

1. Groin No. 1 was 94 ft long, cantilever construction consisting of 664 linear ft of steel sheet piling type MZ-27. The cost was $3,694.84 or $39.24 per foot of groin.
2. Groin No. 2 was 108 ft long, constructed of 240 linear ft of 12-in. timber piles, 216 linear ft of 6 in. by 8 in. timber wales, 108 linear ft of 2 in. by 12 in. oak sheeting. The cost was $4,345.92 or $40.24 per foot of groin.
3. Groin No. 3 was 116 ft long, cantilever construction consisting of 828 linear ft of steel sheet pile, type MZ-32. The cost was $4,972.42 or $42.87 per foot of groin.
4. Groin No. 4 was 130 ft long, constructed of 300 linear ft of 12 in. timber piles, 1080 linear ft of No. 3 gage steel sheeting bolted to 260 linear ft of 8 in. by 2 3/8 in. steel channel wales. The cost was $4,477.20 or $34.44 per foot of groin.

About 500 ft of beach was protected at Camp Perry at a cost of about $35 per linear foot. Information on performance or durability of these groins was not obtained.

The following data on costs is given for other groin construction in the state of Ohio on Lake Erie:

1. At Painesville Township Park, Lake County in September 1950, a groin was constructed of precast-concrete blocks placed in a pyramidal shape over a core of quarry-run sandstone and capped with concrete. The 160 ft of groin cost $29,233.03 or $182.70 per foot.
2. At same location and time as the preceding, one groin was constructed concrete blocks, measuring 2-ft by 4-ft by 6-ft, placed in a pyramidal shape over a quarry run stone core with concrete upper core, capped with blocks set in concrete. The 90 ft of groin cost $6,323.54 or $70.25 per foot.
3. At Avon Lake Village Park, Lorain County in September 1951, a groin was constructed of concrete blocks measuring 2.5-ft by 5-ft by 6-ft with the matching faces of the blocks serrated to prevent displacement. 151 ft of groin (including placed concrete bank connection) was $9,928 or $65.75 per foot.
4. At Lakeview Park, Lorain County in October 1951 a groin was constructed of sandstone blocks. The toe stones weighed 5 tons to 8 tons and the slope stones 4 tons to 5 tons. The toe and slope stones were placed over a 25 lb to 300 lb broken stone core. The structure was capped with stone blocks. The 207 ft of groin cost $20,286 or $98 per foot.

At Selkirk Shores State Park, New York on Lake Ontario a stone groin was constructed in 1953. It was constructed of 5,000 tons of limestone with specific gravity of 2.70 obtained from quarry about 30 miles from the park. The 220 ft of groin cost $32,600 or $107 per foot of groin.

INVESTIGATIONS UNDERWAY AND PROPOSED

Investigations on groins known by the writer to be underway (as of 1961) are being accomplished by the U. S. Army Corps of Engineers. Probably the most urgent study is one under the Chief of Engineers Civil Works Investigation (CWI) Program being performed by the Beach Erosion Bord (BEB) in their model coastal basin. This study considers the effect of waves of various heights and periods and directions on littoral drift along a straight reach of beach. Later single groins and systems of groins will be installed to determine their effects on the beach and on the drift. Various studies under the CWI program that indirectly effect groin design are being carried on at the U. S. Army Engineer Experiment Station (WES) at Vicksburg, Mississippi, and at the BEB. At WES there are studies on wave force on breakwaters and stability of rubber mound breakwaters, the results of which will also apply to rubble mound groins; and a study of shoaling processes. At BEB there are studies on structural design of shore structures; methods of bypassing sand; and criteria for artificial beaches.

Much additional research is needed for the Great Lakes to permit more effective and economical design. Wave statistics have been compiled from hindcasts for stations on Lakes Michigan, Erie, and Ontario. Statistics are needed for Lakes Superior and Huron, and for additional stations on the other three lakes. Prototype wave observations are needed for various locations on all the Great Lakes. Information on durability of groin materials in fresh water and the development of new materials, such as plastics, that would be less expensive and of sufficient strength and durability to withstand wave pressures and weathering would be of great value.

Much has been accomplished in the understanding of the variations in lake levels, and of temporary fluctuations, since 1950. However, there is much yet to do, including the gathering of additional statistics and analysis thereof. It has been previously stated that design still-water level is usually on the order of the elevation of the monthly mean lake level of 20 yr frequency with a 1 yr frequency temporary fluctuation superimposed upon it. The chance of the simultaneous occurrence of this combination is not 100%, but because general storminess usually accompanies high lake levels, it is reasonable to assume such for design purposes. Further study might be warranted as to the frequency of these combined simultaneous occurrences. It appears that the matter of groin spacing might be given consideration for further research. Such research should not only include the shape of the principal portion of the fillet, but also the determination of the length of shore affected by a groin. It would also be of value to determine further information on pattern of filling of single groins and systems of groins and the effect of spacing and height on filling patterns.

CONCLUSIONS

Impermeable groins have been the most widely used means of attempting to stabilize the shore line or create a beach in the Great Lakes area. The majority of the groins were constructed by private interests of wood, steel, stone, concrete, or combinations thereof, using many and varied designs. In many cases no coordinated plan of protection has existed and design of many of the structures have been detrimental. In very few instances has the placement of beach fill accompanied the groin construction. Impermeable groins have been found effective where the littoral drift is adequate to maintain the system. In areas of low littoral drift, groins have not accumulated a beach and have contributed little to the protection of the upland. In many instances, groins have aggravated the erosion effects in adjacent areas, requiring added protective works that might not otherwise have been necessary. However, even in cases in which the downdrift shore is harmed, or when insufficient beach material is collected to provide protection to the backshore, in most cases, benefit is received updrift of the groin by a reduction in the rate of erosion. For maximum benefit to the protected area and minimum damage to downdrift areas, except under specific and probably infrequent conditions, groins should be accompanied by concurrent placement of beach fill in the Great Lakes area. In general, design of groins for the Great Lakes differs little from that for ocean shores, except in the method of determining elevation.

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TABLE 9.—FEDERALLY MADE COOPERATIVE BEACH EROSION CONTROL STUDIES

House Document No., Congress and Session	Title
	Lake Michigan
348, 84th Congress, 2nd Session	Manitowoc Co. from Two Rivers to Manitowoc, Wis. (Jan. 1955)
273, 84th Congress, 2nd Session	City of Kenosha, Wis. (June 1954)
88, 83rd Congress, 1st Session	Racine Co., Wis. (Dec. 1951)
- - -	City of Racine, Wis. (Dec. 1951)
28, 83rd Congress, 1st Session	Illinois Shore of Lake Michigan (June 1949)
- - -	Berrien Co. Mich. (Nov. 1956)
	Lake Erie
177, 79th Congress	Marblehead to Michigan State Line, Lake Erie
	Ohio Shoreline of Lake Erie
126, 83rd Congress, 1st Session	Appendix IV, Sandusky Bay.
32, 83rd Congress, 1st Session	Appendix VI, Sandusky to Vermilion
229, 83rd Congress, 1st Session	Appendix VIII, Vermilion to Sheffield Lake Village
127, 83rd Congress, 1st Session	Appendix XIV, Sheffield Lake Village to Rocky River
- - -	Cities of Cleveland and Lakewood
324, 83rd Congress, 2nd Session	Appendix XI, Euclid to Chagrin River
596, 81st Congress, 2nd Session	Appendix IX, Chagrin River to Fairport Hbr.
351, 82nd Congress, 2nd Session	Appendixes III, VII, XII, Fairport to Ashtabula
350, 82nd Congress, 2nd Session	Appendixes V and X, Ashtabula to Pa. State Line
231, 83rd Congress, 1st	Presque Isle Peninsula, Erie, Pa.
Design Memo	Presque Isle Peninsula, Erie, Pa. (May 1955)
	Lake Ontario
138, 84th Congress, 1st Session	Hamlin Beach State Park, N.Y. (July 1954)
- - -	Braddock Bay, N. Y.
- - -	Fair Haven Beach State Park, N.Y. (Feb. 1954)
- - -	Selkirk Shores State Park, N.Y. (June 1958)
Design Memo	Selkirk Shores State Park, N.Y. (May 1958)
- - -	Niagara County, N. Y.

APPENDIX.—READING REFERENCES

1. "Low Cost Shore Protection for the Great Lakes," by E. F. Brater, Norman Billings, and Dale W. Granger, Lake Hydr. Lab. Research Publications No. 3, Univ. of Mich., Ann Arbor, Mich., 1952.

2. "Geologic History of Great Lakes Beaches," by Jack L. Hough, "Source Materials for Lake Michigan Beaches," by Wm. E. Powers, "The Motion of Sediment Along the South Shore of Lake Erie," by Howard J. Pincus, "Effect of Ice on Shore Development," by James H. Zumberge and James T. Wilson, Proceedings, Fourth Conf. on Coastal Engr., Council on Wave Research, 1952.

3. "Variations in Great Lakes Levels," U. S. Lake Survey, Corps of Engrs., 1952.

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DEEP WATER WAVE GENERATIONS BY MOVING WIND SYSTEMS[a]

By Basil W. Wilson,[1] F. ASCE

SYNOPSIS

This paper concerns the mathematical justification of a procedure for forecasting wave heights and periods within traveling fetches of variable wind over deep water. The method applies to a fixed wind direction along which both the fetch and the wind magnitude are considered to be variable with time. It adapts observational data on significant heights and periods of waves generated by winds of constant velocity to the general case in which the velocity has a dependency on distance. The graphical procedure devised to handle this situation was originally verified only for stationary fetches of variable wind but is now validated for the general case of the moving fetch. The graphical integrations are here replaced by step-wise numerical integrations, readily performed on a high speed digital computer. A direct comparison made between a machine-run wave hindcast and an earlier graphical one for the occasion of the August, 1915 hurricane, that assailed Galveston, Texas, shows good correspondence.

INTRODUCTION

The most recent trends in ocean wave analysis and forecasting have very largely been focused on the development and use of concepts of the wave

Note.—Discussion open until Ocotber 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 2, May, 1961.

[a] Presented at the April, 1960 Amer. Geophysical Union Meeting at Washington, D. C.

[1] Prof. of Engrg. Oceanography, Agricultural & Mechanical College of Texas, College Sta., Tex.

energy spectrum at a point in space and time, descriptive of a partially or fully arisen sea (1),(2),(3),(4),(5),(6).[2] In this country adaptation of these concepts to ocean wave forecasting has been made by W. J. Pierson, G. Neumann and R. W. James, F. ASCE (3), and has come to be known widely as the PNJ method. Quite a radical departure is involved in this procedure from that of the previously existing (so-called) SMB-method that evolved from the work of H. U. Sverdrup and W. H. Munk (7) and of Charles L. Bretschneider, M. ASCE (8),(9) and uses the significant wave as the statistical description of sea condition.

Comparisons made between actual wave recordings and results from application of the two methods have yielded differences in many cases (10),(11),(12),(13),(14),(15) and reasonable agreement in others (16),(17). At least part of the differences can probably be ascribed to the subjective techniques used in determining applicable wind velocities and fetches and in allowing for the changes of these parameters with time, but there is still some controversy about the acceptability or the universality of the Neumann spectrum on which the PNJ method depends (17),(18),(19),(20),(21),(22),(23),(24),(25),(26),(27).

It is not the purpose of the present paper to enter upon an evaluation of the relative merits or faults of these or other wave forecasting techniques. In the final analysis all such techniques have a fundamental dependence on observational data relating wave height, H, and period, T, with steady wind velocity, U, and fetch, x, however precise or ill-defined these quantities may be. Such observational data, evaluated on the basis of the significant wave, as in the SMB forecasting method, are utilized here in a new way in an effort to increase the precision of wave forecasting and eliminate the subjectivity of determining fetch and wind duration. Though Munk (23) has perhaps rather prematurely denounced the significant wave approach, Bretschneider (9), J. R. D. Francis (15) and others, including the writer, are convinced of its engineering usefulness. A feature that is often overlooked is that there is now an impressive volume of observational field and laboratory data that afford a reasonably consistent picture of the empirical relationships governing the involved dimensionless parameters $g\,H/U^2$, $g\,T/U$ and $g\,F/U^2$, when wind duration can be regarded as unlimited. It is the fairly solid edifice of these relationships that supports the structure of the method to be described.

EMPIRICAL RELATIONSHIPS GOVERNING DEEP-WATER GENERATION OF WAVES BY WIND OF CONSTANT VELOCITY

Fig. 1 is taken from the writer's first paper on this subject (28) and is based on an earlier compilation of wave data by Bretschneider (8). It plots the dimensionless wave parameters c/U and $g\,H/U^2$ versus a wind parameter $g\,x/U^2$, in which c is the deep-water velocity of significant waves of height H generated by a steady wind of constant velocity U acting for an unlimited time over a finite fetch x over deep water. The term F denotes the fetch length, in feet; the symbol g has the usual meaning of acceleration due to gravity; likewise the significant wave is by usual definition the wave that has the average height of the highest third of all waves encountered in a representative time of measurement. Bretschneider (9),(29) has since revised his curves by more careful curve-fitting and by establishment of more accurate

[2] Numerals in parenthesis refer to corresponding items in Appendix II.—Bibliography.

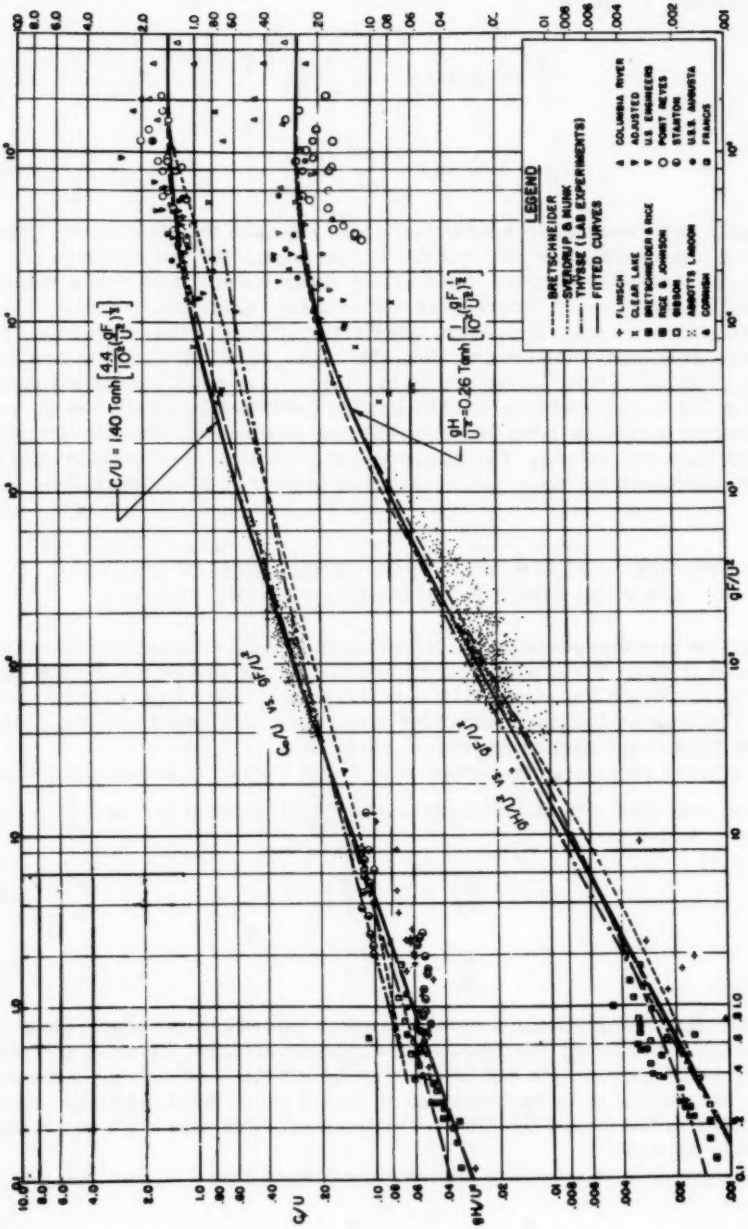


FIG. 1.—RELATIONSHIPS OF WIND-WAVE PARAMETERS FOR CONDITIONS OF UNLIMITED DURATION

ultimate limits, but the changes are not drastic and the previously fitted equations (Fig. 1), namely

$$\frac{gH}{U^2} = 0.26 \tanh\left[\frac{1}{10^2}\left(\frac{gx}{U^2}\right)^{\frac{1}{2}}\right] \quad \ldots\ldots\ldots\ldots (1a)$$

and

$$\frac{c}{U} = 1.40 \tanh\left[\frac{4.36}{10^2}\left(\frac{gx}{U^2}\right)^{\frac{1}{3}}\right] \quad \ldots\ldots\ldots\ldots (1b)$$

still apply with reasonable approximation. In any case the retention of these empirical relationships is not critical to the arguments that are to be presented, yet serves the purpose of permitting comparison of new developments with earlier work. In any future work it should not be difficult to revise Eqs. 1 to fit more closely the latest observational data of the type of Fig. 1.

It has previously been shown (28) how these equations could be used to develop families of forecasting curves in a three-quadrant arrangement such as that of Fig. 2, and how these could be used to forecast graphically the waves generated by a moving fetch of wind within which wind velocity was variable with both time and distance. The theoretical implications of this method were not then discussed but are now the subject of inquiry in this further development.

DERIVED RELATIONSHIPS FOR GENERATION OF WAVES BY WIND WHICH IS VARIABLE OVER THE FETCH

The first problem posed is that of finding the general relationships governing c and H when U is an arbitrary function of x (the distance along the line of fetch) for which Eqs. 1 would be special cases if U(x) happened to be constant. The velocity U(x) is assumed for the present to be steady or invariable with the time at any particular point in the fetch.

To achieve this we differentiate Eqs. 1 with respect to x, regarding U as constant, and then eliminate the parameter $\frac{gx}{U^2}$ in terms of c/U or $\frac{gH}{U^2}$, using Eqs. 1, to arrive at

$$\frac{dH}{dx} = f_1\left(\frac{gH}{U^2}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (2a)$$

and

$$\frac{dc}{dx} = f_2\left(\frac{c}{U}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (2b)$$

In effect this transformation hereby ensures that the incremental gains in wave height and velocity over elemental distances are independent of position and dependent only on the net energy input from the wind. The requirement that U be constant may be discarded in Eqs. 2 and general dependency on x substituted in the form U(x). Simple verification of this procedure is demonstrated in Appendix I.

It is convenient to introduce for simplification of Eqs. 1

$$X = \frac{gx}{U^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3a)$$

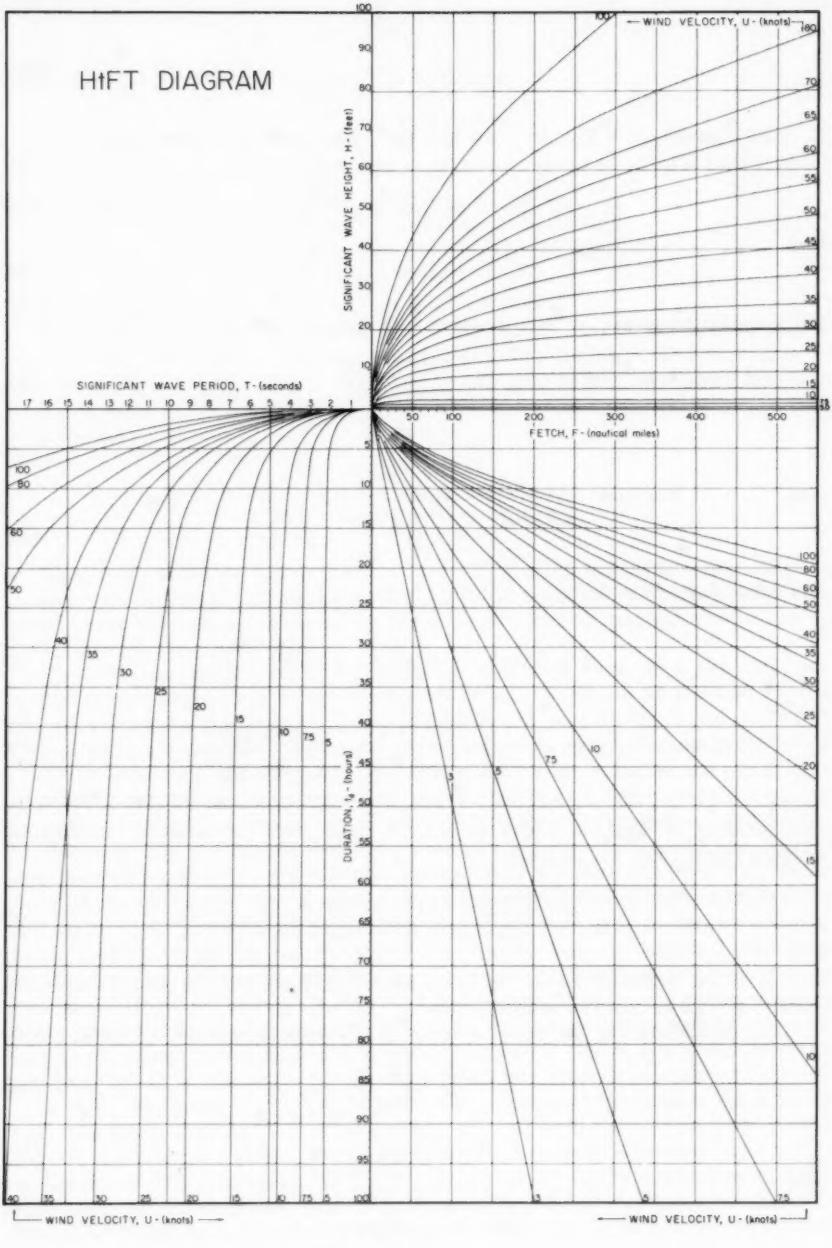

HtFT DIAGRAM
SIGNIFICANT WAVE HEIGHT, H - (feet)
WIND VELOCITY, U - (knots)
SIGNIFICANT WAVE PERIOD, T - (seconds)
FETCH, F - (nautical miles)
DURATION, t_d - (hours)
WIND VELOCITY, U - (knots)
WIND VELOCITY, U - (knots)

FIG. 2.—RELATIONSHIPS (BASED ON FIG. 1) FOR GRAPHICALLY FORECASTING SIGNIFICANT HEIGHTS AND PERIODS OF WAVES IN DEEP WATER

$$Y = \frac{gH}{U^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3b)$$

$$Z = \frac{c}{U} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3c)$$

$k_1 = 10^{-2}$ and $k_2 = 4.36 \times 10^{-2}$ so that the equations are rendered as

$$Y = 0.26 \tanh\left(k_1 X^{\frac{1}{2}}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (4a)$$

and

$$Z = 1.40 \tanh\left(k_2 X^{\frac{1}{3}}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (4b)$$

Differentiation of Eq. 4a with respect to X then yields

$$Y' = \frac{dY}{dX} = \frac{dH}{dx} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5a)$$

$$Y' = \frac{2 k_1^2 Y}{\phi \sinh \phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5b)$$

and

$$\phi = 2 k_1 X^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5c)$$

But from Eqs. 4a and 5c

$$\phi = 2 \tanh^{-1}\left(\frac{Y}{0.26}\right) = \log\left(\frac{0.26 + Y}{0.26 - Y}\right) \quad \ldots\ldots\ldots\ldots (6)$$

while

$$\sinh \phi = \frac{2 \tanh \frac{\phi}{2}}{1 - \tanh^2 \frac{\phi}{2}} = \frac{2 (Y/0.26)}{1 - (Y/0.26)^2} \quad \ldots\ldots\ldots\ldots (7)$$

Substitution of Eqs. 6 and 7 in Eq. 5b, thus gives the desired relationship comparable to Eq. 2a, namely

$$Y' = \frac{2.338 (0.26 - Y)(0.26 + Y)}{\log (0.26 + Y) - \log (0.26 - Y)} \quad \ldots\ldots\ldots\ldots (8)$$

In Eq. 8 the value of $k_1 (10^{-2})$ has been introduced and the units adjusted to render dH/dx in feet per nautical mile.

Differentiating Eq. 4b with respect to X, regarding U as constant, yields

$$Z' = \frac{dZ}{dX} = \frac{U}{g} \frac{dc}{dx} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9a)$$

$$Z' = \frac{8}{3} k_2^3 \frac{Z}{\psi^2 \sinh \psi} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9b)$$

and

$$\psi = 2 k_2 X^{\frac{1}{3}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9c)$$

But from Eq. 4b

$$\psi = \log\left(\frac{1.40 + Z}{1.40 - Z}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

while

$$\sinh \psi = \frac{2\,(Z/1.40)}{1 - (Z/1.40)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

that, substituted in Eq. 9b together with the value of $k_2\,(4.36 \times 10^{-2})$, gives

$$Z' = \frac{7.893 \times 10^{-5}\,(1.40 + Z)(1.40 - Z)}{\left[\log(1.40 + Z) - \log(1.40 - Z)\right]^2} \quad \ldots\ldots\ldots\ldots (12)$$

It will be seen from Eq. 9a that Z' is dimensionless.

Eqs. 8 and 12 are the appropriate differential equations, comparable to Eqs. 2 that now permit U being regarded as a function of x. The functions Y' and Z' take the forms shown in Fig. 3. Fig. 3(a) is for significant height H and Fig. 3(b) is for significant wave velocity c in deep water, with distance x along the fetch.

SPACE-TIME NETWORK OF WIND VELOCITIES FOR A GIVEN FETCH-LINE

The next problem concerns the use of Eqs. 8 and 12 in evaluating the height and period of waves generated ab initio from any arbitrary point in a space-time wind-field of variable wind $U(x, t)$ such as might be encountered along a particular fetch-line.

To visualize such a situation it is convenient to consider a practical example in the movement of the severe hurricane of August 15 through August 17, 1915, across the Gulf of Mexico towards the port of Galveston in Texas. Fig. 4 shows a synoptic sequence of surface pressure patterns at intervals of 12 hr that indicate the progress of the hurricane towards the coast. The particular fetch-line of interest is along the hurricane track from the southeast in a straight line bearing directly on Gilchrist, just north of Galveston.

Shown in Fig. 4 at fixed arbitrary points along the fetch-line AB are arrows and numbers defining respectively the direction and velocity (in knots) of the wind at the different times. (Arrows with numbers indicate wind directions and velocities in knots.) These were computed from the pressure patterns by methods outlined elsewhere (30), (31). The components of the wind velocities directed along AB were derived and plotted in the form of a space-time wind-field that is shown in Fig. 5 appropriately contoured for component wind velocity in increments of 2.5 knots. The shaded or positive zones of wind are components directed towards Gilchrist; white areas define components of wind velocity in the opposite direction, away from Gilchrist.

It will be noted from Fig. 4 that the hurricane track coincided initially with the fetch-line AB but subsequently veered to the left. The node-line in Fig. 5, defining the boundary between white and shaded areas to left-center of the wind-field, corresponds to the path of the eye up to a distance of about 430 nautical miles from Gilchrist. Thereafter, as would be expected, progressively higher winds are encountered along the fetch line in the shaded zone of the wind field.

In Fig. 5 are shown wave height, wave period, and wave propagation lines starting from two arbitrarily selected points A and B in the positive wind-field and radiating into NE, SW, and SE quadrants respectively in relation to these individual points. These (full-line) curves have been derived by the graphical technique (28) in a particular study of Gulf hurricanes (30),(31). The graphical technique used the curves of Fig. 2 in what was essentially a graphical integration process based on Eqs. 1, in lieu of mathematical integration of Eqs. 8 and 12. In adapting the system to a purely mathematical approach, the first concern is in identifying the growth of wave height, H,

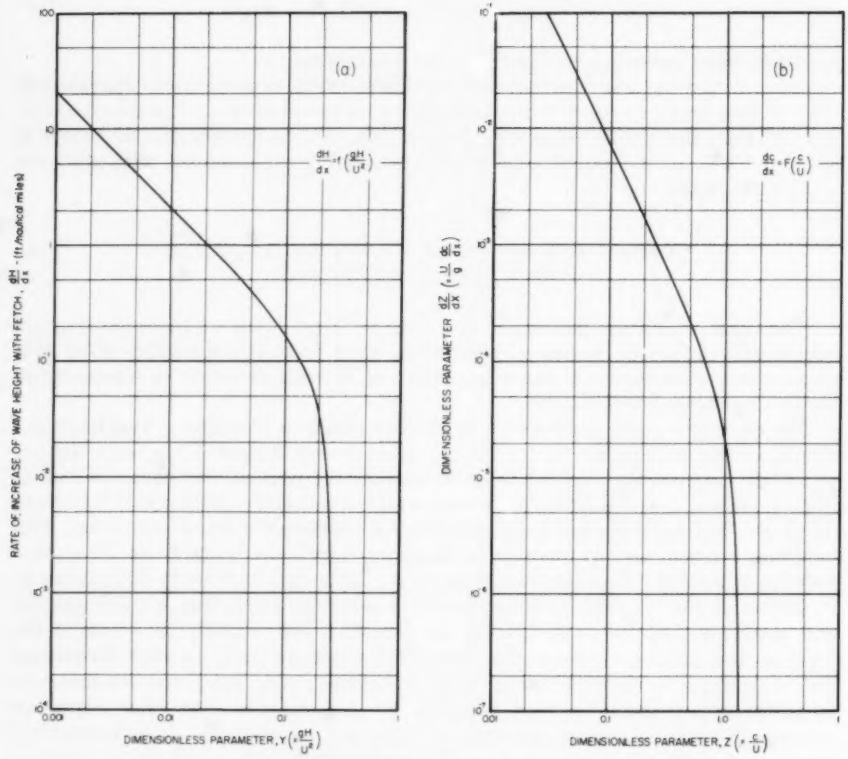


FIG. 3.—RELATIONSHIPS GOVERNING RATES OF CHANGE OF SIGNIFICANT FACTORS

velocity, c, and period, T, over a small initial increment of time or distance, from any arbitrary point in space and time at which waves are considered to be created as ripples.

ELAPSED TIME FOR WAVELETS TO GROW IN A SMALL INITIAL FETCH; Δx_o

Although it is known that ripples or capillary waves have a minimum velocity of about 23 cm per sec and a rate of progression (group velocity)

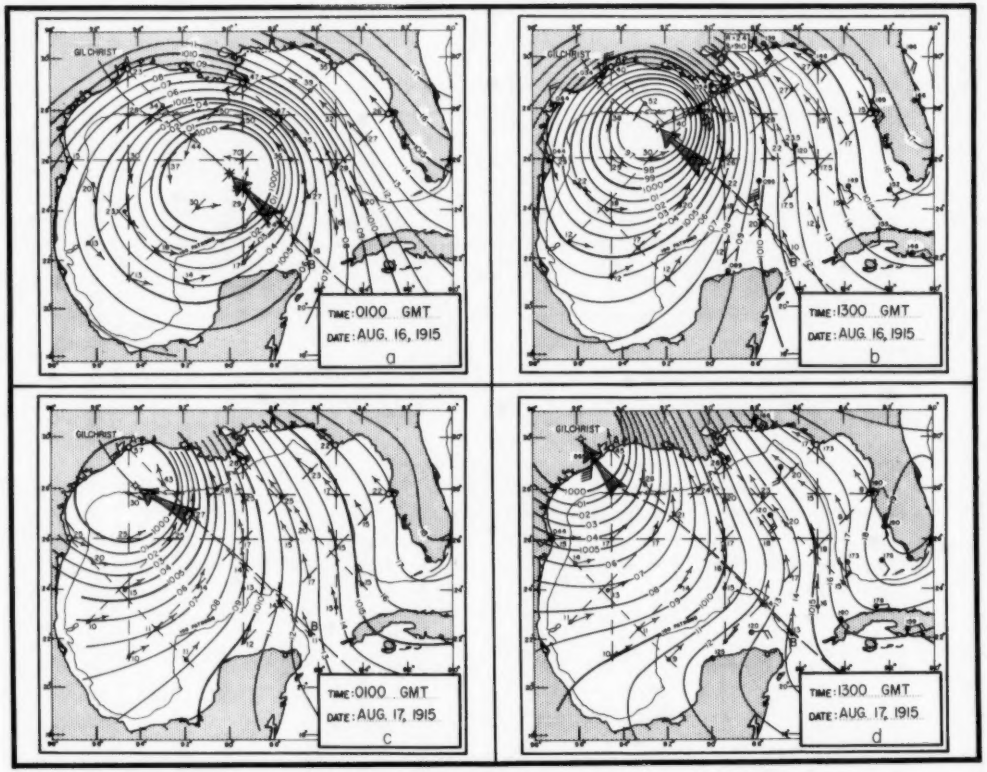


FIG. 4.—12-HOURLY SYNOPTIC PRESSURE PATTERNS FOR THE GULF OF MEXICO, SHOWING THE GALVESTON HURRICANE OF AUG. 16-17, 1915

greater than the velocity of the individual ripples, the empirical nature of the theory of wave growth, here treated, makes the assumption essentially that the wavelets start with zero velocity and continuously and progressively build up to the maximum size that the wind magnitude, fetch, and duration will allow. Further, transition from the capillary to the gravity wave stage is considered to be sufficiently rapid for capillary effects to be neglected in the interval Δx_o, being effected in only a small fractional part of the interval.

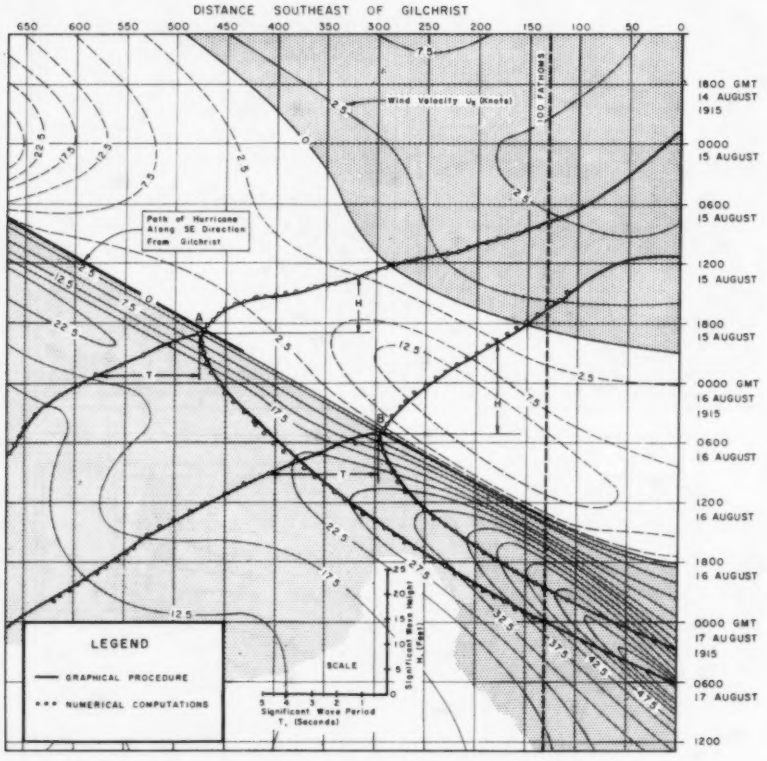


FIG. 5.—SPACE-TIME WIND-FIELD FOR THE GALVESTON HURRICANE ALONG AN APPROACH DIRECTION SOUTH-EAST OF GILCHRIST, TEX.

By choosing a fetch Δx_o sufficiently small and by supposing the wind acting over this distance to be virtually constant in velocity, Eq. 1b may be approximated by

$$\frac{c}{U} \approx \frac{6.10}{10^2} \left(\frac{g\,x}{U^2}\right)^{\frac{1}{3}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

on the basis that gx/U^2 is a small quantity. For $x = \Delta x_o$ at the end of the short fetch, the wave velocity c_a, in knots, will thus be

$$c_a = 2.498 \left(U_o \Delta x_o\right)^{\frac{1}{3}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

in which U_o is measured in knots and Δx_o in nautical miles.

The elapsed time for the wavelets to attain the end of the fetch Δx_a may be found as

$$\Delta t = \int_0^{\Delta x_o} \frac{2\,dx}{c} \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

This supposes that Δt, is very much longer than the time taken for the wavelets to emerge from the capillary phase, and that over the time Δt_o the effective rate of progression is at the group velocity $c/2$ applicable to deep-water gravity-waves. On substituting Eq. 13 in Eq. 15 and integrating, the elapsed time, in hours, is found to be

$$\Delta t_o = \left[1.20 \frac{\left(\Delta x_o\right)^2}{U_o}\right]^{\frac{1}{3}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

for U_o in knots and Δx_o in nautical miles.

If Δt_o is selected as 1 hr, that experience would suggest is adequate, Δx_o is obtained readily from Eq. 16, as

$$\Delta x_o = 0.761 \sqrt{U_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

for U_o in knots. This value inserted in Eq. 14 yields, for U_o in knots,

$$c_a = 2.281 \sqrt{U_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

HEIGHT AND PERIOD ATTAINED BY WAVELETS AT THE END OF A SMALL INITIAL FETCH, Δx_o

On the basis that $\frac{gx}{U^2}$ is small, Eq. 1a reduces to

$$\frac{gH}{U^2} = \frac{0.26}{10^2} \left(\frac{gx}{U^2}\right)^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

At the end of the small fetch Δx_o, therefore, the height H_a, in feet, of the wavelets is found from Eq. 19 as

$$H_a = 6.036 \times 10^{-2} U_o \sqrt{\Delta x_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

for U_o in knots and Δx_o in nautical miles.

For the particular case that Δt_o is taken equal to 1 hr, Eq. 17 may be inserted in Eq. 20 to give

$$H_a = 5.265 \times 10^{-2} U_o \sqrt[4]{U_o} \text{ ft.} \quad \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Because the period T of short gravity waves of low height in deep water is defined sufficiently accurately by the formula

$$T = \frac{2\pi}{g} c \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

it follows from Eq. 14 that the period T_a, in seconds, of the wavelets at the end of the fetch Δx_o will be

$$T_a = 0.824 \left(U_o \Delta x_o\right)^{\frac{1}{3}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

for U_o in knots and Δx_o in nautical miles.

Again, for the special case of Δt_o equal to 1 hr, Eq. 17 may be inserted in Eq. 23 to give

$$T_a = 0.753 \sqrt{U_o} \text{ secs} \quad \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

for U_o in knots.

NUMERICAL INTEGRATION OF WAVE PARAMETERS IN A VARIABLE SPACE-TIME WIND-FIELD

For a wind field such as that of Fig. 5 in which U(x, t) is a complicated function of both distance x and time t the only satisfactory means of integrating Eqs. 8 and 12 is by a numerical step-wise procedure. In schematic form this is indicated in Fig. 6 in respect of just the wave propagation-line. The numerically integrated wave-path of wind-generated waves in deep water and the elements of the numerical computations are shown in Fig. 6.

The wind-field is there represented as a rectangular space-time lattice whose intervals in distance, λ, and time, τ, are taken sufficiently small to ensure good accuracy in the integrations. It is supposed that the wind velocity components along the given fetch direction are known at all lattice intersection points, being derivable from the contoured wind field. Any intersection point of the lattice may be chosen arbitrarily as a starting point for the propagation of waves. Whether distance, λ, or time, τ, is used as differential increment of integration depends essentially on the value of the group-velocity V of the waves, which for wind waves in deep water is

$$V \approx \frac{c}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

Because the gradient of the propagation line in the lattice (Fig. 6) is a measure of V it is clear that τ is the differential increment of integration if

$$c/2 < \frac{\lambda}{\tau} \quad \ldots\ldots\ldots\ldots\ldots\ldots (26a)$$

and λ is the differential increment of integration if

$$c/2 > \frac{\lambda}{\tau} \quad \ldots\ldots\ldots\ldots\ldots\ldots (26b)$$

As shown in Fig. 6, τ is the differential increment of integration in the initial step, and therefore by making $\Delta t_o = \tau$ in Eq. 16 the increment of distance, Δx_o, is at once determined, as also are c_a, H_a and T_a from Eqs. 14, 20, and 23, respectively. If τ is suitable chosen as 1 hr, Eqs. 17, 18, 21, and 24 apply. The value of U(x, t) used in these equations is that, (U_o), pertaining to the starting point 0.

From hereon, further integration invokes use of Eqs. 8 and 12. The wind velocity U_a for the next step, starting at a, (Fig. 6), must be interpolated from the lattice-point values $U_{o,1}$ and $U_{1,1}$ in which subscripts refer to the lattice numbers taken in the order of absissa, ordinate. Thus,

$$U_a = U_{o,1} + \frac{\Delta x_o}{\lambda}\left(U_{1,1} - U_{o,1}\right) \quad \ldots\ldots\ldots\ldots (27)$$

is a linear interpolation that is usually sufficiently accurate if τ and λ have been well chosen. It is necessary to apply the test, Eq. 26, in the next interval

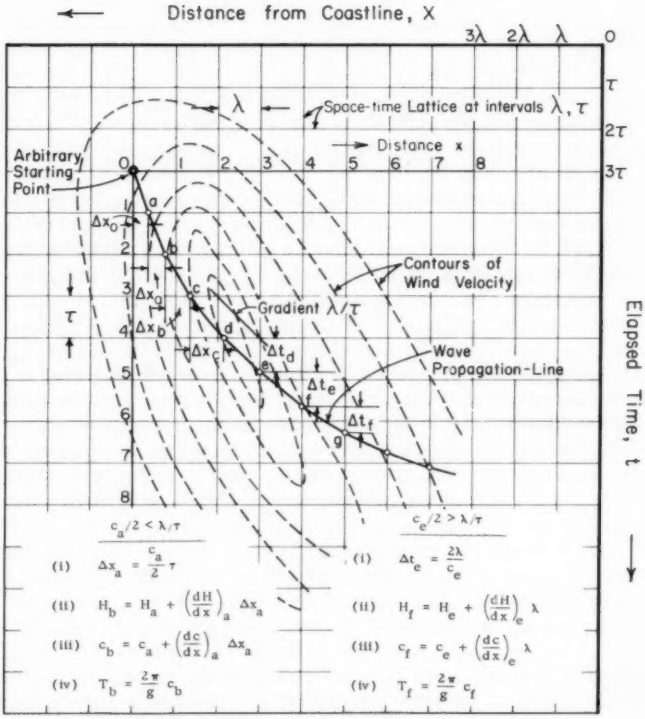


FIG. 6.—SCHEMATIC REPRESENTATION OF WIND-FIELD SPACE-TIME LATTICE

using c_a. Thus, if $c_a/2 < \lambda/\tau$ the time interval τ is used to compute

$$\Delta x_a = \frac{c_a}{2}\tau \quad \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

establishing the point b in Fig. 6.

To compute H_b and c_b, use is made of Eqs. 8 and 12. Thus in Eq. 8, by inserting

$$Y_a = \frac{gH_a}{U_a^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (29a)$$

$$Y'_a \left[= \left(\frac{dH}{dx}\right)_a \right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (29b)$$

is found, leading to

$$H_b = H_a + \left(\frac{dH}{dx}\right)_a \Delta x_a = H_a + Y'_a \Delta x_a \quad \ldots\ldots\ldots\ldots (30)$$

Similarly, from Eq. 12 for

$$Z_a = \frac{c_a}{U_a} \quad \ldots\ldots\ldots\ldots\ldots\ldots (31a)$$

$$Z'_a \left[= \frac{U_a}{g} \left(\frac{dc}{dx}\right)_a \right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (31b)$$

is found and c_b follows from

$$c_b = c_a + \left(\frac{dc}{dx}\right)_a \Delta x_a = c_a + \frac{g Z'_a}{U_a} \Delta x_a \quad \ldots\ldots\ldots\ldots (32)$$

The period T_b of the waves at b is found directly from Eq. 22, namely

$$T_b = \frac{2\pi}{g} c_b \quad \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

At this stage it becomes necessary to interpolate the wind velocity U_b in anticipation of the next step. If $(\Delta x_o + \Delta x_a) < \lambda$, as shown in Fig. 6,

$$U_b = U_{0,2} + \frac{\Delta x_0 + \Delta x_a}{\lambda} \left(U_{1,2} - U_{0,2}\right) \quad \ldots\ldots\ldots\ldots (34)$$

The procedure previously outlined may be continued following a search at each step to ensure that $c/2 < \lambda/\tau$. If, as at d, (Fig. 6), the test shows $c_d/2 > \lambda/\tau$, then a change over is necessary from use of τ to use of λ as the differential increment of integration. Thus, in preparation for this,

$$V_d = \frac{c_d}{2} = \frac{x_3 - x_d}{\Delta t_d} \quad \ldots\ldots\ldots\ldots\ldots\ldots (35)$$

from which

$$\Delta t_d = \frac{2\left(x_3 - x_d\right)}{c_d} \quad \ldots\ldots\ldots\ldots\ldots\ldots (36a)$$

and

$$\Delta x_e = x_3 - x_d \quad \ldots\ldots\ldots\ldots\ldots\ldots (36b)$$

The computation of H_e, c_e and T_e presents no difficulty, following the pattern of Eqs. 32 and 33, but the interpolation of U_e would now have to accord with

$$U_e = U_{3,4} + \frac{\Delta t_d}{\tau} \left(U_{3,5} - U_{3,4}\right) \quad \ldots\ldots\ldots\ldots\ldots (37)$$

In computing H_f, and c_f and all subsequent values of height and velocity, the differential increment of integration would be λ, as in the following:

$$H_f = H_e + Y'_e \lambda \quad \ldots\ldots\ldots\ldots\ldots\ldots (38a)$$

and

$$c_f = c_e + \frac{gZ'_e}{U_e}\lambda \quad \ldots\ldots\ldots\ldots\ldots\ldots (38b)$$

COMPARISON OF GRAPHICAL AND NUMERICAL PROCEDURES

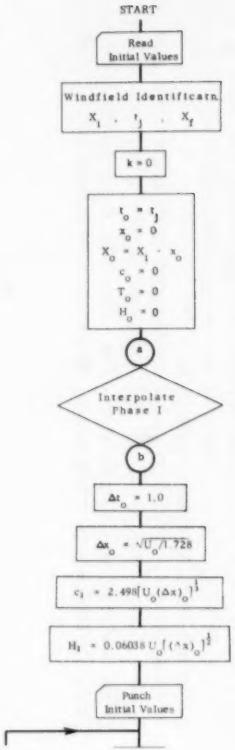
The whole numerical procedure is quite readily programmed for operation on a high-speed digital computer (Fig. 7). The results of a pilot computation applied to the points A and B in the wind field of Fig. 5 are shown by the series of plotted points closely following the (full-line) height, period, and propagation lines as obtained by the graphical procedure. The satisfactory check thus secured is both a verification of the graphical method and a demonstration of the practicability of the mathematical method.

Whereas the time taken for the graphical construction of wave height, period and propagation lines for a single starting point, such as A or B in Fig. 5, normally required from 15 min to 20 min, the corresponding time for their computation numerically on a No. 650 IBM electronic computer using fixed point decimal notation was reduced to from 20 sec to 30 sec. This advantage of speed opened up the possibility of investigating many more propagation lines from wind-field areas than could normally be contemplated by the graphical method.

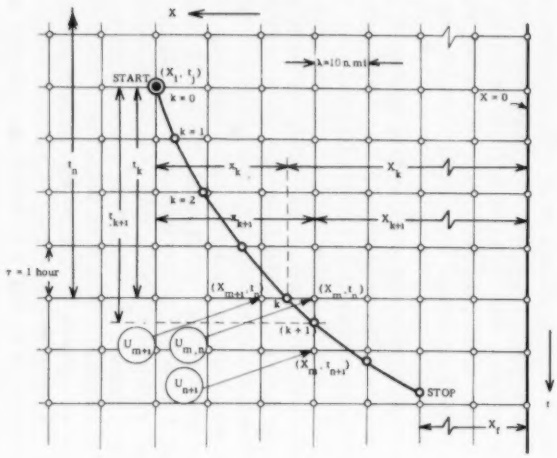
In an extended study of the waves generated by the 1915 hurricane from the south-east off Gilchrist, the wind-field of Fig. 5 was prepared as a lattice of values of wind velocity component, U, with intervals $\lambda = 10$ nautical miles and $\tau = 1$ hr, as shown in Fig. 8. Numerical integrations were run from lattice intersection points within a band of the wind-field defined by the 10-knot wind velocity contour as the upper limit and the 20-knot contour as the lower limit. Representative samples of wave propagation lines from within this area are shown in Fig. 8 carried as far as the 100 fathoms depth line that defined approximately the edge of the continental shelf, 130 nautical miles SE of Gilchrist (see inset, Fig. 8 that shows the hurricane track).

The comparable graphical study for this area had to be satisfied with only five propagation lines outside the 100 fathom depth, as shown in Fig. 9, starting from arbitrarily selected points close to the upper boundary of the positive (shaded) wind zone, so chosen to ensure the longest possible domination of the waves by wind. The heights and periods of the significant waves at the 100-fathom depth are recorded in Fig. 9 in which the propagation lines intersect the depth-line. These values are embodied in the dash-line envelope curves of Fig. 10, depicting the maximum significant wave heights and periods likely to have reached the continental shelf-edge at the times shown. These results are adapted from a previous work by the writer (30). The paucity of information for defining the envelope curves as obtained graphically required supplementation with intelligent guessing.

That the graphical estimates were quite good is shown by the very much more precise definition of the envelope curve made possible by the high speed numerical computations (Fig. 10). The zone of the wind-field within which computations were made, however, was insufficiently wide to give the

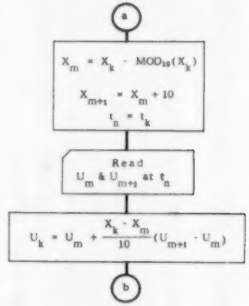


KEY TO WINDFIELD & NOTATION

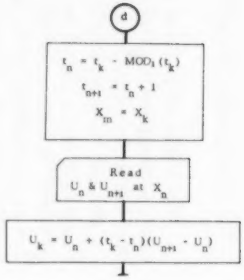


SUB-ROUTINES

INTERPOLATION : PHASE I



INTERPOLATION : PHASE II



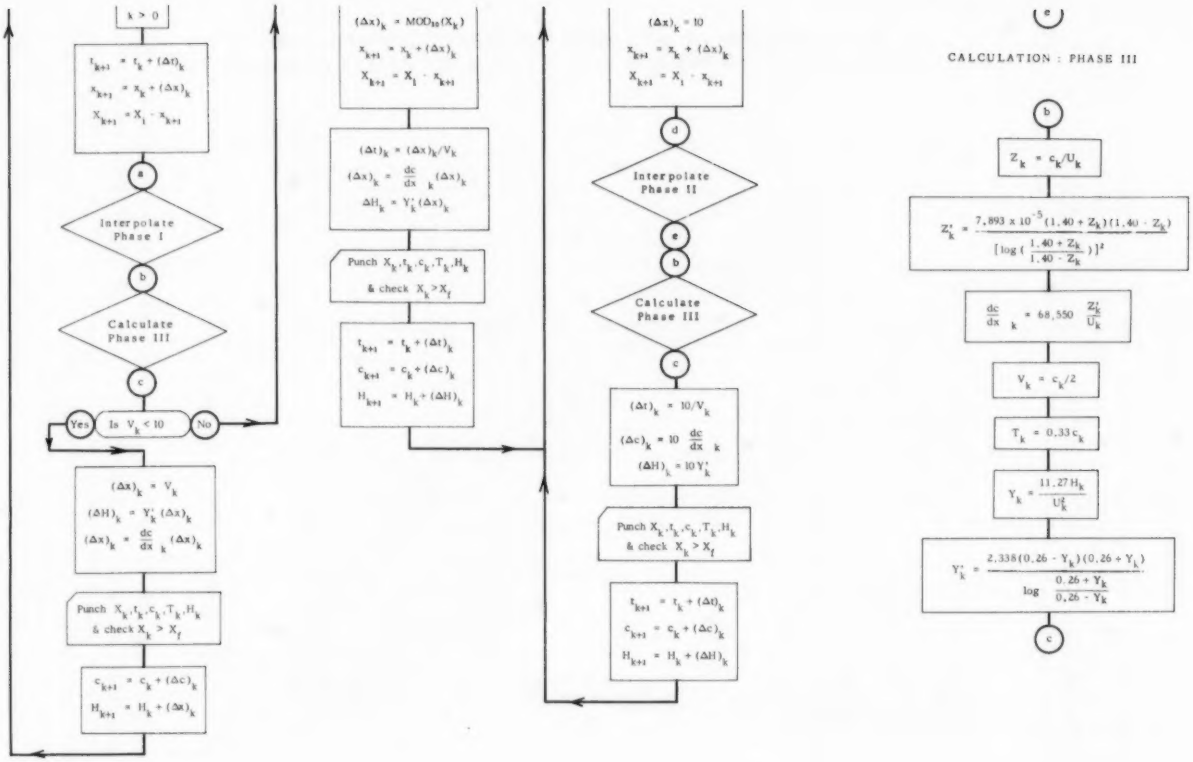


FIG. 7.—FLOW DIAGRAM FOR NUMERICAL WAVE PREDICTION

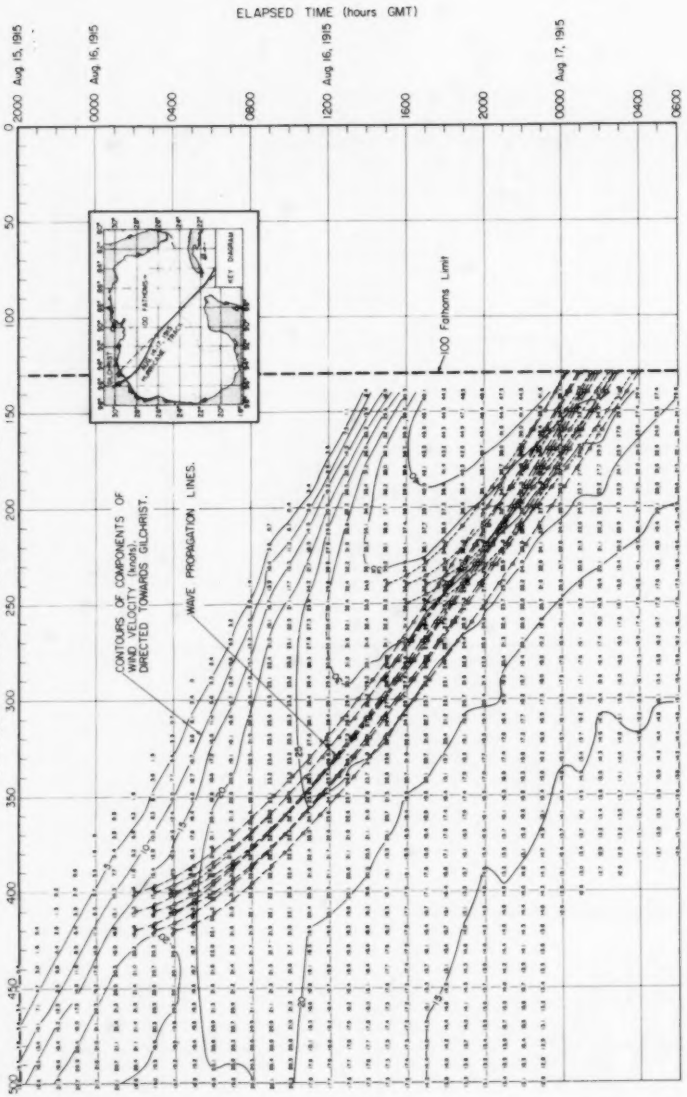


FIG. 8.—WIND-FIELD SPACE-TIME LATTICE FOR THE GALVESTON HURRICANE ALONG THE SOUTH-EAST APPROACH DIRECTION TO GILCHRIST, TEX.

computed envelope curves any definition earlier than 1800 August 16, 1915, or later than 0600 on August 17.

SUMMARY AND CONCLUSIONS

The description of a sea condition at any point in space and time can usually be interpreted in terms of a wave energy spectrum or of the size of the prevalent significant wave, that connotes a recognized statistical distribution of incident wave heights and periods. The latter description is the basis

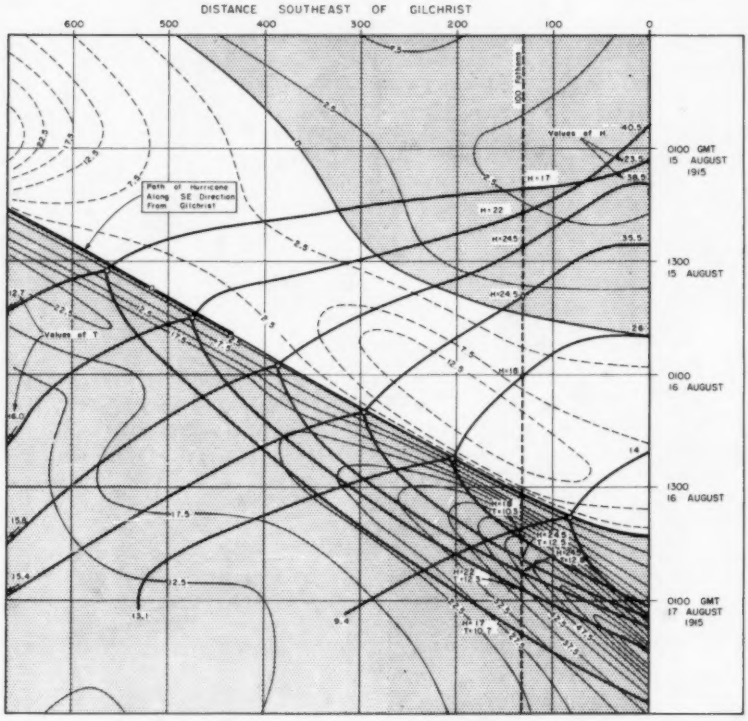


FIG. 9.—SPACE-TIME WIND-FIELD FOR THE GALVESTON HURRICANE ALONG THE SOUTH-EAST APPROACH DIRECTION TO GILCHRIST, TEX.

for the developments in this paper. Observational data, linking heights and periods of significant waves with wind velocity and fetch under conditions of steady wind of constant magnitude over deep water, are used to evolve empirical relationships of wide scope that define the differential rates of increase of wave height and velocity when the wind velocity is variable with distance. These differential equations are suitably derived from empirical formulas

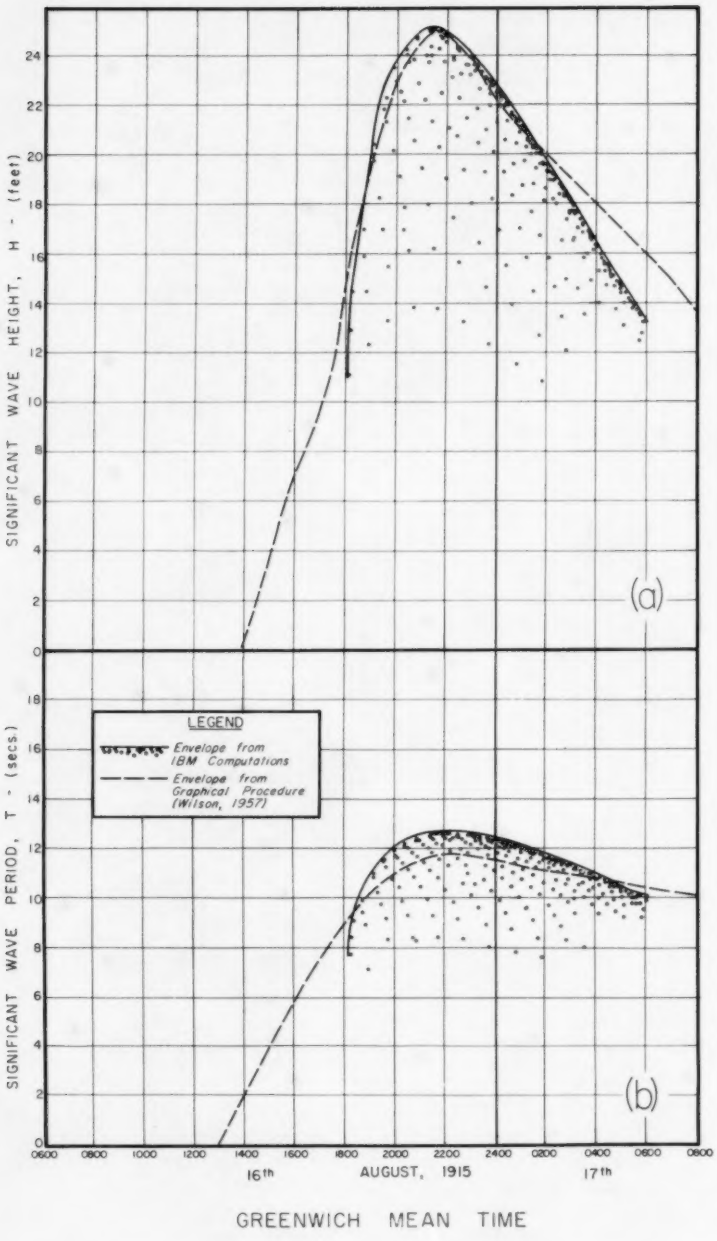

SIGNIFICANT WAVE HEIGHT, H - (feet)
SIGNIFICANT WAVE PERIOD, T - (secs.)
(a)
(b)
LEGEND
Envelope from IBM Computations
Envelope from Graphical Procedure (Wilson, 1957)
0600 0800 1000 1200 1400 1600 1800 2000 2200 2400 0200 0400 0600 0800
16th AUGUST, 1915 17th
GREENWICH MEAN TIME

FIG. 10.—COMPARISON OF GRAPHICAL AND NUMERICAL PREDICTIONS OF SIGNIFICANT FACTORS

fitted to the observational data and in effect generalize the latter for variable wind.

In the general case of wind blowing in a particular direction over an expanse of water, the wind velocity may vary not only with distance but also with time, particularly if the wind system is moving, as in a hurricane or cyclonic storm. The changes in wind velocity bearing in the given direction may best be visualized in a wind-field or space-time plot of velocity components. Such a wind-field is shown for the SE approach direction to Gilchrist, Texas, on the occasion of the severe hurricane of August 15 through August 17, 1915, that attacked Galveston from the Gulf of Mexico. The waves generated by this hurricane along the SE direction bearing on Gilchrist were investigated in earlier work by a graphical procedure that in effect integrated the differential equations without defining the latter but by using forecasting curves based on the observational data.

In the present paper the equations and procedures are developed for integrating the differential equations by numerical step-wise methods, that are readily programmed for execution by fast electronic computers. A high speed digital computer was used in pilot computations to check the graphical wave hindcasts for the wind-field mentioned. In this work it was found appropriate to use wind-field lattice spacings of $\lambda = 10$ nautical miles and $\tau = 1$ hr as the differential increments for integration. Successful checks between the graphical and methematical methods were obtained.

A machine computation, covering the history in growth of height, period, and travel of waves originating from a specific point in space and time within the wind field, could be performed in from 20 sec to 30 sec as against a time of some 15 minutes to 20 minutes required for the graphical operation. The use of faster computers already promises a reduction of the time to something of the order of a second or two. As a result of the great saving in time and effort, it becomes feasible to compute large numbers of wave propagation lines covering needed areas of wind-field, thereby permitting sharp definition of envelope curves of maximum significant wave height and period, plotted as functions of time, at any point along the given direction.

The need for such envelope curves is prescribed by the multiplicity (even infinity) of wave propagation lines that can reach any point along the fetch over a period of time. Clearly, the highest significant wave, of longest period, arriving at a specific time over-rides any lesser values. The statistical nature of the significant wave concept ensures that a proper distribution of individual wave heights and periods exists, within which any inferior significant wave heights and periods can be accommodated. To some extent this deduction has to be intuitive because it is impossible to convey an adequate physical picture of the mechanism by which waves are continuously generated from ripples at all points along the fetch, as is hypothesised in this forecasting method.

Although the numerical procedure here described may now be considered to verify and supplant the earlier graphical method of treating wave generation in moving fetches of variable wind, there is still great need for rigorous comparison of its predictions with actual wave measurements. The only real test that has been made is that of P. S. Shrivastiva (32) this was not very accurately performed, yet gave results comparable with the SMB and PNJ forecasting methods. Application of the graphical method in studies of hurricane waves in the Gulf of Mexico (30), (31) has nevertheless confirmed that rational

and consistent results can be expected. Research, being carried out in 1961, is attempting to give final validation to the method.

ACKNOWLEDGMENT

This work has been pursued under grants from the Engineering Foundation of New York, N. Y., the Humble Oil and Refining Company, the Mobil Oil Company and through the sponsorship of the Office of Naval Research, Contract No. N7 onr 48702 (Project NR 083--36). The writer is grateful to R. O. Reid for a demonstration, that was the key to Appendix I and the justification of Eqs. 2, and to R. Kilmer who programmed and conducted the numerical computations on the No. 650 IBM computer in accordance with the flow diagram of Fig. 7.

APPENDIX I.—DETERMINATION OF A FUNCTION $H^2 = f_2[x, U(x)]$ FROM A KNOWLEDGE OF THE FUNCTION $H^2 = f_1[x, U_0]$

The problem posed is that of determining a function $H^2 = f_2[x, U(x)]$ from a knowledge of the specific value of this function when the independent variable U is constant, or independent of x, as in $H^2 = f_1(x, U_0)$.

We may differentiate the known function f_1, partly with respect to x and find

$$\frac{\partial(H^2)}{\partial x} = \frac{\partial f_1}{\partial x} = f(x) \quad \ldots\ldots\ldots\ldots\ldots\ldots (39)$$

As shown in Fig. 11, this yields the general equation of the curve of intersection of a vertical plane parallel to the reference plane $\left(\frac{d(H^2)}{dx}, x\right)$, and the curved surface $\frac{d(H^2)}{dx} = \frac{d[f_2(x, U)]}{dx}$ in this three-dimensional representation of of the problem.

By resorting to the known function $H^2 = f_1[x, U_0]$ the variable x may be eliminated on the right hand side of Eq. 39, to yield

$$\frac{d(H^2)}{dx} = F(U) \quad \ldots\ldots\ldots\ldots\ldots\ldots (40)$$

in which U (replacing U_0) is now the independent variable. This in effect has established the equation of any particular curve of intersection of a vertical plane parallel to the reference plane $\left(\frac{d(H^2)}{dx}, U\right)$ and the surface $\frac{d(H^2)}{dx} = \frac{d[f_2(x, U)]}{dx}$ as shown in Fig. 11.

If we are concerned with the value of H^2 attained when U is a specific function of the variable x, namely

$$U = \theta(x) \quad \ldots\ldots\ldots\ldots\ldots\ldots (41)$$

it will be seen from Fig. 11 that the curve of intersection of the vertical surface through $U = \theta(x)$ and the curved surface $\frac{d(H^2)}{dx} = \frac{d[f_2(x, U)]}{dx}$ is

$$\frac{d(H^2)}{dx} = F[U(x)] \quad \ldots\ldots\ldots\ldots\ldots\ldots (42)$$

This curve of intersection may be projected without change of the Eq. 42 on to the reference plane $\left(\frac{d(H^2)}{dx}, x\right)$ thereby defining an area whose value is the required general function:

$$H^2 = f_2[x, U(x)] = \int_0^x F[U(x)]\,dx \quad \ldots\ldots\ldots\ldots (43)$$

In illustration of this procedure, with particular reference to the problem of this paper, we may suppose that it is known from observation that the re-

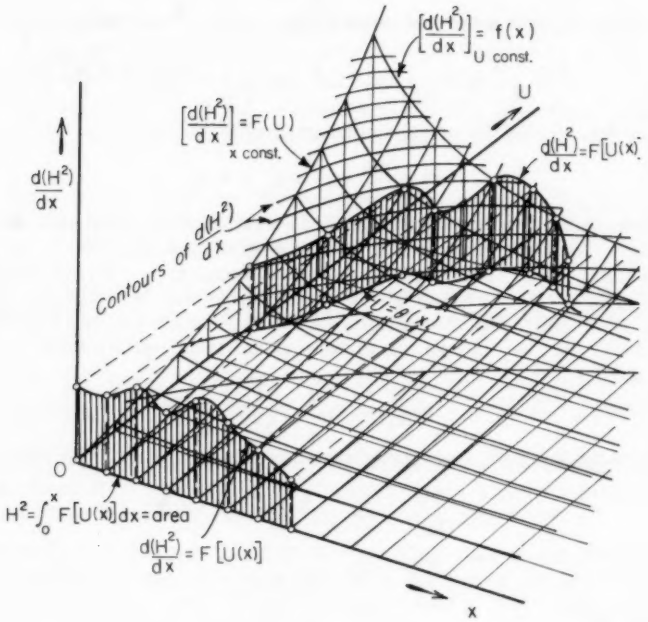


FIG. 11.—SCHEMATIC REPRESENTATION OF THE DETERMINATION OF A GENERAL FUNCTION (H^2) OF TWO VARIABLES (x AND U) FROM A KNOWN FUNCTION IN WHICH ONE OF THE VARIABLES (U) IS CONSTANT

lationship between significant wave height H, fetch x, and constant wind velocity U_0, agrees with the equation

$$\frac{g\,H}{U_0^2} = k\left(\frac{g\,x}{U_0^2}\right)^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (44)$$

in which k is a numerical coefficient. On squaring this

$$H^2 = K U_0^2 x \quad \ldots\ldots\ldots\ldots\ldots\ldots (45)$$

if

$$K = k^2/g \quad \ldots\ldots\ldots\ldots\ldots\ldots (46)$$

Eq. 45 thus is the known function, $f_1(x, U_0)$. Differentiating is partially with respect to x, in conformity with Eq. 39,

$$\frac{\partial(H^2)}{\partial x} = K U_0^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (47)$$

This is actually independent of x, without requiring the use of Eq. 45 for its elimination; it is therefore of the form of Eq. 40 and may be written

$$\frac{d(H^2)}{dx} = K U^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (48)$$

If now the wind velocity is arbitrarily taken as a linear function of x, namely

$$U(x) = U_0 + a\ x \quad \ldots\ldots\ldots\ldots\ldots\ldots (49)$$

in which both U_0 and a are constant, then, in accordance with Eq. 43

$$H^2 = \int_0^x K(U_0 + ax)^2\, dx = Kx\left(U_0^2 + U_0 ax + \frac{a^2x^2}{3}\right) \quad \ldots\ldots (50)$$

By use of Eq. 49, Eq. 50 may be converted readily to

$$H^2 = \frac{Kx}{3}\left(U^2 + U_0 U + U_0^2\right) \quad \ldots\ldots\ldots\ldots\ldots (51)$$

This then would represent the general relationship governing wave height dependency on wind fetch x and variable wind velocity U(x) in terms of the original observation. If U were constant and equal to U_0, Eq. 51 would reduce identically to the observation under which Eq. 44 was formulated.

As a final check on the validity of the procedure we note that the differential of Eq. 43 must satisfy the mathematical requirement that

$$\frac{d(H^2)}{dx} = \frac{\partial f_2}{\partial x} + \left(\frac{\partial f_2}{\partial U}\right)\left(\frac{dU}{dx}\right) \quad \ldots\ldots\ldots\ldots\ldots (52)$$

By differentiating Eq. 51 partially with respect to x, first, and then U,

$$\frac{\partial f_2}{\partial x} = \frac{K}{3}\left(U^2 + U_0 U + U_0^2\right) \quad \ldots\ldots\ldots\ldots\ldots (53a)$$

and

$$\frac{\partial f_2}{\partial U} = \frac{Kx}{3}\left(2U + U_0\right) \quad \ldots\ldots\ldots\ldots\ldots (53b)$$

From Eq. 49,

$$\frac{dU}{dx} = a \quad \ldots\ldots\ldots\ldots\ldots\ldots (54)$$

Accordingly, on inserting Eqs. 53 and 54 in Eq. 52, there results

$$\frac{d(H^2)}{dx} = \frac{K}{3}\left(U^2 + U_0 U + U_0^2\right) + \frac{K}{3} ax\left(2U + U_0\right) = KU^2 \quad \ldots\ldots (55)$$

that is identical with Eq. 48, as it should be.

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APPENDIX III.—NOTATION

Symbols adopted for use in this paper are presented here for ease of reference and for the aid of discussers:

a = dimensional constant in Eq. (A 10);

$a, b, c, d \ldots$ = serial subscripts referring to incremental steps of integration (see Fig. 6);

c = velocity in deep-water of significant waves;

F = function of variables;

f = function of variables;

f_1, f_2 = specific forms of f;

g = acceleration due to gravity;

H = height of the significant wave;

i = integer subscript of distance ($= 0, 1, 2, 3 \ldots$);

j = integer subscript of time ($= 0, 1, 2, 3 \ldots$);

K = dimensional constant $[= k^2/g]$;

k = (1) dimensional constant [Eq. (A 6)]; (2) integer subscript (Appendix B) ($= 0, 1, 2, 3 \ldots$);

k_1, k_2 = numerical coefficients [Eq. (3)];

$\log$ = natural logarithm (to base e);

m = integer subscript (Appendix B), $(=0, 1, 2, 3 \ldots)$;

$MOD_1(t_k)$ = defines the fraction by which the number t_k exceeds its nearest integer;

$MOD_{10}(X_k)$ = defines the fraction of 10 by which the number X_k exceeds 10 p in which p is the largest integer for which $10p < X_k$;

n = integer subscript (Appendix B), $(=0, 1, 2, 3 \ldots)$;

T = period of significant waves;

t = variable time;

t_j = particular value of $t[=j\tau]$;

t_k = value of t elapsed to the kth step from the start of the digital computation (Appendix B);

t_{k+1} = value of t elapsed to the (k + 1)th step from the start of the digital computation (Appendix B);

t_n = particular value of $t[=n\tau]$; either $t_n = t_k$ or $t_n = t_k - MOD_1(t_k)$

t_{n+1} = particular value of $t[=(n+1)\tau]$;

Δt = incremental length of time;

$(\Delta t)_k$ = particular value of $\Delta t\ [= t_{k+1} - t_k]$;

U = component of surface wind velocity along a line of fetch;

U_o = (1) constant value of U (Appendix A); (2) initial value of U at space-time lattice point (X_i, t_j);

U_k = value of U at space-time lattice point (x_k, t_k) or $(X_k, t_j + t_k)$;

U_m = value of U at space-time lattice point (X_m, t_n);

U_{m+1} = value of U at space-time lattice point (X_{m+1}, t_n);

U_n = value of U at space-time lattice point (X_m, t_n);

U_{n+1} = value of U at space-time lattice point (X_m, t_{n+1});

$U(x)$ = component surface wind velocity along a line of fetch, variable over the fetch;

V = group-velocity of significant waves in deep water;

X = (1) dimensionless parameter $[= gx/U^2]$; (2) distance from coastal station along given fetch line;

X_k = value of X (2) defining the position $x_k\ [= X_i - x_k]$;

X_m = particular value of $X\ [= m\lambda]$; either $X_m = X_k$ or $X_m = X_k - MOD_{10}(X_k)$;

X_{m+1} = particular value of $X\ [= (m+1)\lambda]$;

x = variable distance or fetch over which the wind blows;

x_k = value of x from the start to the k-th step of the digital computation (Appendix B);

x_{k+1} = value of x from the start to the (k + 1)-th step of the digital computation (Appendix B);

Y = dimensionless parameter $\left[= gH/U^2 \right]$;

Y' = differential coefficient of Y with respect to X(1);

Z = dimensionless parameter [= c U];

Z' = differential coefficient of Z with respect to X(1);

Δx = incremental length of fetch;

$(\Delta x)_k$ = particular value of $\Delta x \left[= x_{k+1} - x_k \right]$;

$\theta(x)$ = function of the variable x;

λ = interval of distance in space-time lattice;

π = universal constant (3.14159 . . .);

τ = interval of time in space-time lattice;

ϕ = dimensionless parameter defined by Eq. (5iii); and

ψ = dimensionless parameter defined by Eq. (9iii).

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DISCUSSION

Note.—This paper is a part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 2, May, 1961.

BREAKWATERS IN THE HAWAIIAN ISLANDS[a]

Discussion by M. H. Marty and E. E. Chapus

M. H. MARTY[13] and E. E. CHAPUS,[14] A. M. ASCE.—The paper is of particular interest in that it gives general consideration to the many problems that have to be dealt with by engineers faced with the complex difficulties associated with unique site conditions.

It is understandable that the author did not deal with all the problems he mentions as fully as they deserve, because it would be beyond the scope of the paper. It is felt that further development of some points will prevent unjustified general conclusions being drawn from discussions of a limited number of particular cases. A few such points that are considered to be of particular importance will be discussed herein.

Repairs to Kahului Breakwaters.—

Damage.—Attention should be drawn to the fact that 603 tetrapods were used for the repairs and this should be kept in mind in connection with the statement that 3 tetrapods were broken and 30, that is, only 5%, were rolled away. It may be assumed that some of these tetrapods could be salvaged and reinstated.

As far as tetrapods were concerned, damage was confined to the breakwater heads. No damage occurred along the straight portions on which tetrapods were used. The main damage was to an older section of breakwater in which no tetrapods were used.

Design.—It should be emphasized that this damage was expected and that it was allowed for when the design was decided on. Two points of view were expressed, one by M. H. Marty, who recommended that a fully preliminary study be made and who stated that the proposed design would be incapable of withstanding local wave conditions if the proposed layout and tetrapod weights were used. The other viewpoint was that of the engineers in charge of the project who intended to use a lighter armor layer after having computed that the cost of periodic repairs would be less than that of constructing a breakwater capable of standing up to normal conditions at Kahului.

The latter viewpoint prevailed. The damage that occurred was part of the accepted risk, and experience may be considered to have confirmed the cost estimates because damage was localized and did not endanger the breakwater as a whole.

Calculated risk.—It is considered that the concept of risk has not been fully exploited because before taking a risk it is necessary to know what the risk is and to have calculated it.

---

[a] June 1960, by R. Q. Palmer (Proc. Paper 2507).

[13] General Mgr. Sotramer, France.

[14] Pres. Neyrpic, Inc., New York, N. Y.

No model study was undertaken in connection with Kahului before the breakwaters were repaired. The author makes it clear that the likely extent of damage was only realized after the repairs had been completed.

In spite of the comparatively inaccurate prototype data, it is felt that a model study specifically concerned with Kahului would have provided more reliable information regarding the risks that were being taken when constructing breakwaters incapable of withstanding the most severe conditions.

It seems, therefore, that the concept of risk ought to be replaced by that of calculated risk.

An essential prerequisite to breakwater construction would appear to be model studies carried out at the design stage. These would make it possible to estimate the damage associated with each wave height and to determine the most economic design on the basis of the latter.

If this viewpoint is accepted, damage must not be assessed on an absolute basis but rather in comparison with the amount of damage that has wittingly been accepted as being permissible.

The terms "minor failure" or "substantial damage" have no meaning unless the damage they refer to is compared with the damage that was expected and that economic considerations made acceptable.

The concept of calculated risk appears to be particularly important and ought to be used much more widely.

Although this concept was not applied fully at Kahului, because no computations were made beforehand, it seemed that some emphasis had to be laid on the basis on which the designs for the repair work were developed.

The Use of Tribars.—The author's analysis of the first project involving Tribars gives rise to further questions insofar as economic problems are associated with the purely technical problems arising in the laboratory and on construction sites.

It would be extremely helpful if the author could amplify the data given in his paper so that general laws, or at least conclusions applicable to particular cases might be derived from data provided by the Nawiliwili project. It would be possible to do this if, in addition to the figures tabulated in his paper, the author were to give some additional details.

Economic data.—It would be interesting to know the actual cost of repairing the breakwater heads and the straight section of the breakwater. It appears that the original design was modified as follows during construction:

1. the end slope was made flatter;
2. tribars were used to replace stone removed to facilitate construction;
3. some of the tribars were reinforced; and
4. only one layer of tribars was used on the straight portion of the breakwater instead of two layers because of the shallow water.

Technical data.—In Fig. 13, the tetrapod armor layer has been wrapped around the breakwater head whereas in Fig. 12 the tribar armor layer covers a smaller area. Why was the tetrapod armor layer extended in this manner if the stone on the harbor side of the breakwater was sufficiently stable? Will it be possible to use a single layer of nested tribars on the type of breakwater normally encountered, that is, breakwaters on which the armor layer has to be taken much further below water level than at Nawiliwili?

Finally, a very important point, to which the author has paid much attention, has to be considered.

After several tests had been carried out on full-size tetrapods up to 16 tons in weight, Messrs Daniel, Chapus and Dhaille[15] concluded that reinforcement of precast concrete components of this type, unless very heavy reinforcement is used, gives a false impression of security because both reinforced and unreinforced components break under exactly the same shock conditions, the only difference being that reinforced components do not fall apart. Corrosion resulting from exposure to sea water rapidly nullifies any safety factor that may have been assumed.

The preceding considerations are particularly important when dealing with a single layer of tribars. According to the figure cited by the author, it appears that the stability of the whole layer depends mainly on the tribars being nested together efficiently. If breakage of components reduces their nesting capacity, there will be a danger of the armor layer being rapidly damaged by a sort of chain reaction.

Do systematic tests with tribars bear out this assumption as to what takes place when breakage of reinforced or unreinforced tribars occurs?

It is probably worth while recording that the author's comparison between various types of facing block can only be of a fairly artificial theoretical nature. The necessity of determining the optimum profile of a maritime structure for each given particular case is the important point to consider. There are many examples[16] to show that this optimum profile depends on a multiplicity of factors that may require several major modifications to be made to the theoretical profile determined from general systematic scale model test results. Such modifications might not always be the same for all the different facing block shapes.

Conclusions.—The very large number of problems associated with breakwaters is further added to with the introduction of new devices such as today's tribar and the ten year old tetrapod.

Tetrapods have become standard practice and offer no surprises for those who use them properly. Ten years of experience and the several hundred thousand tetrapods used all over the World are adequate proof of their reliability because the cost of breakwaters, their economic consequences and their effect on everyday life are too important for it to be permissible to overlook any factor.

It is therefore possible to agree with the author when answering the assertion that "you can't built it too strong," he says "Economical breakwaters will be built" strong enough "when criteria for all aspects of design are developed, in order that the desired factor of safety can be assured for each element of the structure."

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15 "Tetrapods and Other Precast Blocks for Breakwaters," Proceedings, ASCE, Vol. 86, No. 3, September. 1960.

16 Ibid., Figs. 10 and 11.

PREDICTION OF MAXIMUM PRACTICAL BERTH OCCUPANCY [a]

Closure by Thomas J. Fratar, Alvin S. Goodman and Austin E. Brant, Jr.

THOMAS J. FRATAR,[10] F. ASCE, ALVIN S. GOODMAN,[11] M. ASCE, and AUSTIN E. BRANT, JR.,[12] A. M. ASCE.—Mr. Hill's thoughful discussion of the application of probability theory to berth occupancy at Hampton Roads brings out some important concepts that are implicit in the theory.

Mr. Hill notes that the Poisson Law applies only when the assumption can be made that all the available vessel berths in a group of berths are equally attractive to all shippers. This is certainly true where vessel berths are leased on an individual basis to a terminal operator (that may be a shipping company, a stevedoring company, a railroad, or the like), because the operator is interested principally in the efficiency and operating capacity of his berth. Under this condition, the attractiveness of a vessel berth is measured by the cost to the user, or the cost of handling cargo at the berth.

At a port in which all vessel berths are actually operated by a port-wide agency, the cost to the user of a terminal will probably be the same regardless of the efficiency and operating capacity of individual berths. Under this condition, if all berths in a port are located within the same general area and have equal accessibility to railroads and highways serving all parts of the port's tributary area, the operational characteristics of all berths need not be the same to obtain maximum efficiency in utilization. In other words, the most efficient berth in a group of berths could be operationally scheduled to handle a much greater proportion of the total commerce of the group than the least efficient berth. Considering the group as a whole, however, the over-all performance could be predicted by the Poisson Law.

With specific application to Hampton Roads, where berths are located in various sections of the port and have considerably different inland transport connections, terminals in all sections of the port must be made equally efficient to be equally attractive to shippers. This illustrates the advantage of imporving the least efficient berths to the level of the more efficient ones as the first step in a portwide development program.

With respect to Mr. Hill's comment that Newport News and Norfolk should be considered as separate ports, it should be noted that the application of the Poisson Law shows that two independent sets of berths have a much lower operating capacity than the combined group. For example, two groups of six berths each have a maximum practical occupancy of 53%, whereas a single

[a] June 1960, by Thomas J. Fratar, Alvin S. Goodman, and Austin E. Brant, Jr. (Proc. Paper 2517).

[10] Partner, Tippetts-Abbett-McCarthy-Stratton, New York, N. Y.

[11] Proj. Engr., Tippetts-Abbett-McCarthy-Stratton, New York, N. Y.

[12] Engr., Tippetts-Abbett-McCarthy-Stratton, New York, N. Y.

group of twelve berths has a maximum practical occupancy of 63%. This increased efficiency can be attained by consolidating operational control of all berths in a port in a single agency and by providing as far as possible equal accessibility for all terminals.

TETRAPODS AND OTHER PRECAST BLOCKS FOR BREAKWATERS[a]

Discussion by T. P. Anantram

T. P. ANANTRAM,[15] F. ASCE.—The authors have correctly stressed the necessity for designing the blocks to resist breakage during manufacture, placing, and during the unavoidable disturbances due to gales and so forth. They have also postulated a non-reinforced wave-breaker block for obvious reasons.

Of the four types of wave-breaker blocks that the authors have illustrated, namely the Stabit, the Tribar, the Hollowblock, and the Tetrapod, the least vulnerable, from a point of view of damage, is the Tetrapod, and the most stable, the Hollowblock. Even so, the Tetrapod is liable to break if what the writer has witnessed during a gale in May, 1959, at Porbandar is a criterion. Five to 10 ton slabs of rock were ripped off the faulty bed in the breakwaters area of the open Arabian Coast and hurled up the beach. Hollowblocks would be more stable but also liable to break.

Considering the precedings, the writer would submit to the authors his suggestion for the design of a block incorporating the tetrapod and hollow-block ideas.

Assume a Tetrahedral Solid of edge, x ft in length. The height would be $\sqrt{6x/3}$ ft. Slice off $\sqrt{6x/18}$ ft. length at the four apices. The four trunkated ends should than be rounded off to a further length of $\sqrt{6x'/18}$ each leaving the solid to rest on any of the four faces that would be a regular hexagon, 1/3 x' side. Round off the four edges. Provide a void of the shape of the tetrapod inside the solid, the hollows sticking out at the centers of the hexagons with an end diameter of 0.2 x'. That will be the proposed wave-breaker block.

The advantages are: 1. Easy lifting by slinging through the holes in the block, 2. Better stability after positioning, 3. Better dissipation of the force of breakers by the outside streams of water criss-crossing the rounded voids between blocks and by the streams inside the blocks nullifying each other and dropping down by gravity through the bottom-most holes, 4. Much less liable to break than the four types illustrated by the authors, 5. More easily manufactured by using deflatable tetrapod forms for the hollows in the blocks and more easily handled.

SHRI M. G. HIRANANDANI,[16] C. V. GOLE,[17] and Y. D. PENDSE,[18].—The authors have stated that the breakwaters at Genoa complied with modern design

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[a] September 1960, by P. Danel, E. Chapus and R. Dhaille (Proc. Paper 2590).

[15] Senior Civ. Engr., M-S Gammon India (P) Ltd., Hamilton House, Ballard Estate, Bombay 1, India.

[16] Dir., Central Water and Power Research Sta., Poona, India.

[17] Dy. Dir. (SG), Central Water and Power Research Sta., Poona, India.

[18] Research Officer, Central Water and Power Research Sta., Poona, India.

standard and should have stood up the storm of 1955 without damage. According to the modern standard methods, the depth at the toe of the vertical face should have been 4 times the height of maximum waves allowing for a factor of safety of 2 to 2.5. In this particular case, namely outer breakwater of a recent construction, the depth was only 10.5 m at the base of the vertical face even though the wave heights as high as 5 m were already observed at the site. Detailed computation were also made to test the stability of this breakwater according to prevalent modern standard practices. This revealed that the breakwater was unstable and did not comply with the modern design standard.

Regarding the use of tetrapods for the armour layer of breakwaters used for Crescent City the authors have stated that 25 ton tetrapods as recommended by them were a little less in weight than given by the general curves. According to the curves given in a paper by the senior author[19] the weight of tetrapods required on a slope of 1 1/2:1 for a wave height of 26 ft, even for a poor stability curve, works out to be slightly less than the one recommended by the authors. The authors have also given curves for "fairly good," "good," and "very good" stability. The difference in weights of tetrapods required for fair and very good stability comes to as much as two times. Will the authors make it clear as to what is the percentage loss of tetrapods allowed for various stability curves? It is believed, the percentage loss is worked out with respect to number of tetrapods in the armour layer. It is also not understood why the tests were carried out with a profile quite different from that existing at the site. It may be pointed out that in the case of theoretical test profile depth at the structure was higher than that actually existing at the site.

Regarding the tetrapods suggested for the Marine Drive Sea Wall at Bombay, the authors carried out tests with a theoretical profile, significantly different from the existing profile. If the tests had been carried out with existing profile the depths at the toe of the breakwater even at high water would not work out to be more than 6 ft, thus preventing the initial breaking of waves of more than 5 ft.

It has been stated by the authors that to suppress the overtopping of the seawall at Marine Drive, larger size tetrapods were recommended. During the monsoon of 1960, however, considerable overtopping was observed even for normal wave heights of 6 ft making it impossible to walk along the seawall-side of foot-path.

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[19] Proceedings, 4th Coastal Conf. on Coastal Engrg., October, 1953, p. 390.

SHARK RIVER INLET SAND BY-PASSING PROJECT[a]

Discussion by Per Bruun

PER BRUUN.[3]—The author wrote that the Shark River Inlet, although dredged to 20 ft depth apparently by-passes some material naturally. It does not appear, however, that any considerable quantity of sand by-passed by natural processes reaches the downdrift shore until it has moved northward a number of miles from the Inlet. Considering a tidal prism Ω of about 4.7×10^6 cu yd, a mean sea level gorge area A of about 600 sq yd, an annual amount of littoral drift M of about 265,000 cu yd based on accumulations (probably less than the total amount of northward drift balance that may exceed 300,000 cu yd), maximum discharge per second during spring tide conditions based on tidal hydraulic computations, Q_m = about 600 cu yd, the inlet stability characteristics listed in Table 2 are obtained. The Ω/M ratio = 18 is far below the value of 150 that seems to be the limiting value for a reasonable natural stability and flushing. The corresponding value of $Q_m/M = 2 \times 10^{-3}$ is much below 10^{-2}, that has a similar importance.[4] The M/Q_{max} ratio is about 450 and inlets with $M/Q_{max} > 200$ to 300 all have a predominant "bar by-passing." So, the author's conclusion regarding bar by-passing as the only possible by-passing method seems to be correct. No definite distance for the return of the possible by-passed material to the downdrift beach is indicated, but "a number of miles" seems to mean 3 miles to 4 miles.

TABLE 2.—INLET STABILITY CHARACTERISTICS

Ω	4.7×10^6 cu yd	$\frac{\Omega}{M}$	~ 18
Q_m	about 600 cu yd per sec	$\frac{Q_m}{M}$	$\sim 2 \times 10^{-3}$
M	265×10^3 cu yd per yr	$\frac{M}{Q_{max}}$	~ 450

In this respect it is interesting to compare certain Shark River Inlet data with data from the Ft. Pierce Inlet on the Florida East Coast. Table 3 lists such data that are supposed to have a similar degree of accuracy as Mr. Angas' Shark River data.

[a] September 1960, by W. Mack Angas (Proc. Paper 2599).

[3] Head, Coastal Engrg. Lab., Univ. of Florida, Gainesville, Fla.

[4] "Stability of Coastal Inlets," by P. Bruun and F. Gerritsen, North Holland Publishing Co., 1960.

From Table 3, it is seen that the Ft. Pierce Inlet, due to its ample tidal flow compared with the annual amount of littoral drift and probably also due to the shape of its inlet entrance that in fact is similar to the entrance of the Shark River Inlet, (long updrift—shorter downdrift jetty) is able to transfer about half of the southward drift (this may be a little on the optimistic side). From figures based on Table 2, it is surprising that the Shark River Inlet is able to by-pass any material at all with a channel dredged to 20 ft depth. It is possible that the special entrance—configuration with the long updrift and the short downdrift jetty is helpful in this respect as it undoubtedly is at Ft. Pierce. Ebb-currents at Shark River Inlet will tend to run NE; and wave action from SE perhaps together with the northward component of the ebb-currents may cause some bot-

TABLE 3.—SHARK RIVER INLET AND FT. PIERCE INLET CHARACTERISTICS

Characteristics (1)	Shark River (2)	Ft. Pierce (3)
Depth of inlet entrance channel	20 ft	30 ft (actual maximum depth)
Length of jetty updrift side	950 ft	2000 ft
Length of jetty downdrift side	625 ft	1200 ft
Quantity of drift	300,000 cu yd	250,000 cu yd
Quantity of drift by-passed naturally	50,000 cu yd (estimate)	125,000 cu yd (evaluated based on surveys)[a]
Downdrift deficiency because of the inlet littoral barrier	250,000	125,000 cu yd
$\frac{Q}{M}$	18	320
$\frac{Q_m}{M}$	2×10^{-3}	15×10^{-3}
$\frac{M}{Q_{max}}$	450	68

[a] "Coastal Engineering Study of Ft. Pierce Beach," Coastal Engrg. Staff, Univ. of Florida, Gainesville, Fla., Vol. XII, No. 9, 1958.

rom creep of material in northward direction. But as mentioned by Mr. Angas the material apparently has trouble returning to shore. A similar phenomenon is probably also in action at Ft. Pierce, at the Vistula River entrance in Poland, at Thorsminde, Denmark, and it is discussed for an improvement of Scheveningen, Holland. It may perhaps in a more complex way also be working at the harbor entrance in Golden Gate, California and at Aveiro, Portugal. In the case of the latter a sandbar on the downdrift (South) side seems to function as receiving station for by-passed material.

The by-passing operation undertaken at Shark River Inlet was based on a trestle, dragline, and truck-practice. Sand was coarse and as stated by the

author F. B. Cogan[5] sand removed had only a negligible amount of fine material, for which reason it is anticipated that refill will have a relatively long life when compared to normal hydraulic beach fill from a borrow area in the lagoon or bay, Mr. Angas said: "It has been the experience of the State that about 25% of a beach fill made by hydraulic methods is quickly lost." This is probably fair statement. Our experience in Florida at this time is similar and

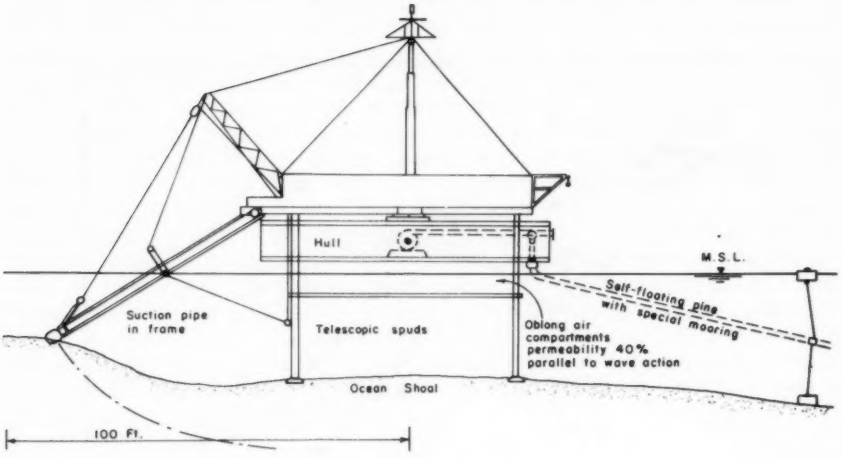

FIG. 18.—TEXAS TOWER DREDGE

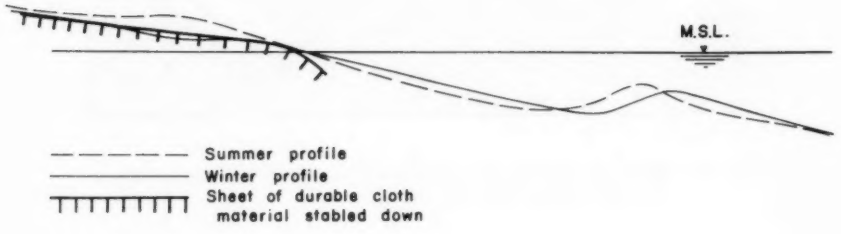

FIG. 19.—BEACH PROTECTION BY SYNTHETIC CLOTH SHEETS

in order to increase our problems it seems to be more and more difficult to find suitable sand for beach nourishment partly because so much sand is being used for bay-fills. Many bays and lagoons simply do not contain good sand anymore, but ample quantities of the so-called "Hawthorne formation" that is

5 "Shore and Beach," Journal, American Shore and Beach Preservation Assoc., December, 1959.

a mixture of sand, silt, and shell. It has been difficult to find sand in the lower Clearwater Bay area on the Gulf Coast and in the Palm Beach and Pompano Beach areas on the lower East Coast. In certain cases we may have to be satisfied although it is known beforehand that about 1/3 of the material placed will be lost immediately after the fill operation. For this reason it may be economically advantageous to truck in suitable fill from landward sources located several miles away even if the price may be 50% higher than hydraulic fill from 2,000 to 5,000 ft borrow areas. From the Florida East Coast there is an example of a unit price of $0.76 based on 5 miles trucking distance. In the

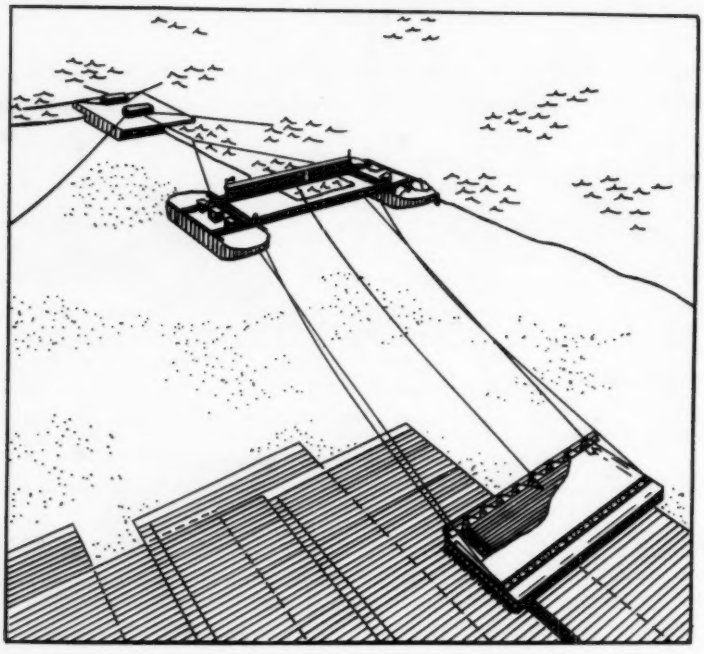

FIG. 20.—OUTLAY OF NYLON SHEETS FOR BOTTOM PROTECTION AT TIDAL GAPS IN HOLLAND

case of the Shark River project that was based on a modest quantity of sand, truck-transport across the inlet apparently was more economical than hydraulic transport. But with much larger quantities as for example, in the case of the Santa Monica Bay project where in 1947 through 1948 15,000,000 cu yd of sand was dumped on 6 miles of beach for a unit price of 22.6¢ per cu yd it is without discussion that hydraulic transport will always prove to be cheapest. Right now (1961) a pump and trestle arrangement is under discussion for the Fire Island Inlet, Long Island. It was earlier suggested by the French Neyrpic at Cotonou in Africa.

The next question is whether the Shark River Inlet by-passing arrangement will prove adequate for the future. Mr. Angas does not go into details of that problem. It may prove adequate when considered as a short-range plan, but long-range it is difficult to imagine that it will be sufficient considering partly the slightly rising sea level and particularly the inevitable loss of material to the offshore bottom. Generally speaking it may be said that by-passing is good, but it is not enough. This comes out clearly in several surveys mentioned in House Documents of recent date by the U. S. Corps of Engineers. Extra supply of sand may be secured by mining either landward (as in California at Santa Monica) or seaward. In the case of the latter the shallow water dredge or perhaps the Texas tower dredge (Fig 18) relying on offshore shoals may be the first step, the submerged dredge or pumping plant the next. But, perhaps other and not very conservative measures of reinforcing the beach in the grand scale will have to be sought. Fig. 19 shows a beach that has been protected against extreme erosion caused by severe storms by putting a sand tight cloth down in it. Such experiments on beach reinforcement are now underway on the Danish North Sea Coast at Thyboroen Inlet based on Dutch nylon sheets used as bottom protection at tidal gaps (Fig. 20). In Florida similar experiments are being conducted that are based on plastic sheets stapled down in the beach by fiberglass staples. At times of extreme storms material "available on the beach" may be washed out in deep water from which it would never return to nourish the beach. If it was not available the situation may be different. It may be possible to replace some of Mr. Angas' coarse sand of excellent quality by sheets of resistant cloth material nailed down in the beach and nearshore offshore bottom.

PNEUMATIC BARRIER[a]

Discussion by T. M. Dick

T. M. DICK.[5]—In the latter part of the paper the author uses Taylor's theoretical equation to obtain the discharge of air necessary to obtain a given upward flow of water.

The writer carried out some model experiments[6] at Queen's University, Kingston, and found that Taylor's equation rather overestimates the velocity. The empirical equation found expressed in the author's symbols is,

$$W_o = 1.2\ (v\ g)^{1/3} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (29)$$

As shown by Eq. 29, the velocity is independent of the depth of submergence of the perforated pipe. However, the discharge of water was found to vary linearly with the depth and can be expressed simply by the equation.

$$q_s = 0.3\ h_s\ (v\ g)^{1/3} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (30)$$

This is the same as Eq. 28 except that the coefficient 0.71 has been replaced by 0.3.

In his conclusions, the author mentions that he anticipates that little trouble will arise from sediment depositing at the pneumatic barrier. However, in the same model investigation mentioned previously, the writer made a few preliminary experiments to determine the possibility of using a screen of air bubbles to inhibit the entrance of silt and sand into a harbour. Powdered coal was used in the tests and it was found that two banks of material were built up, one on each side of the perforated pipe. In general, the effect of the air screen is to throw material into suspension and this is amplified when waves are present.

Once in suspension the sediment is carried outwards by the surface currents and then begins to fall through the water. Where the sediment will eventually deposit, probably depends largely on the fall velocity and the degree of turbulence in the water. One must be careful here when trying to apply the results of model tests to nature as the similitude is not well defined or known.

However, it may prove that in many cases in nature, the bottom sediment may be heavy enough to cause a bar to form on each side of the pipe. In the salt water, flocs that would be heavy enough to settle out may also augment the bar deposits. This bar of course could form even if the pneumatic barrier is only operated during slack river flow.

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[a] September 1960, by Ian Larsen (Proc. Paper 2600).

[5] Dept. of Civ. Engrg., Queen's Univ., Kingston, Ontario.

[6] "A Laboratory Study of Pneumatic Breakwaters," by T. M. Dick and A. Brebner, C. E. Research Report No. 12, July, 1960, Civ. Engrg. Dept., Queen's Univ. at Kingston, Ontario.

Lastly, the author recommends model studies to study the usefulness of the pneumatic barrier in reducing salt infiltration in tidal estuaries.

As mentioned previously, surely the extrapolation of model tests, that use an air stream in water, to the prototype cannot be relied on. If the author has information to the contrary, the writer would be very pleased to learn more about it.

STUDIES OF A CHANNEL THROUGH PADRE ISLAND, TEXAS[a]

Discussion by Per Bruun

PER BRUUN.[2]—Mr. Hansen's very well-written paper is interesting in particular because it tells a story, between the lines, of how things can be planned when a failure results and next explains how a failure can be improved by using artillery so heavy that one is tempted to think of less caliber solutions.

In 1957 an inlet was completed on the littoral drift shore in question that has about a 300,000 cu yd per yr northward balance drift. The paper classifies the inlet as "a tidal inlet" although the tidal part of the water current apparently plays a smaller role only. The inlet completed in 1957 was supposed to be protected against the sand drift but the design of jetties was permeable with top elevation +5 ft above M.L.W. at a place at which wave heights of 5 ft or more are rather common. The blocks were placed on a bottom that, in places, only consisted of a thin layer of sand resting on soft layers deposited in the bay at a time when the barrier was further eastward. Apparently it was realized that some sand would pass through the jetties, but it was hoped that this sand would be flushed away by the inlet currents. Meanwhile with a 100 ft (bottom width) by 10 ft deep bay channel that was supposed to be stable (this means that the maximum velocity had to be below 3 ft/per sec) the current in the dredged 250 ft wide 16 ft deep channel would normally be < 1 ft/per sec. With such low velocity very little flushing ability could be expected. The author stated that "in November, 1957, severe storms produced a period of very heavy seas and severe settlement of the jetties. Scouring currents washed away material at the inner ends of the jetties and resulted in shoreline recession sufficient to flank both jetties. With the effectiveness of the jetties destroyed, extensive shoaling occurred...." It is believed that it is fair to state that said storm only accelerated the deterioration of the inlet entrance because with 50% permeability jetties a balance drift of 3000,000 cu yd (M) and a maximum discharge under normal weather and flow conditions of probably maximum 80 cu yd per sec to 120 cu yd per sec (Q_m), the inlet most likely would not have lasted very long anyhow. Its Q_m/M ratio would be less than 10^{-3} and to obtain a reasonable stability[3] it ought to be $> 10 \times - 10^{-3}$. The M/Q_m ratio is as high as about 3,000 or more and already with a M/Q_m ratio of 200 to 300 material would tend to by-pass the inlet on a shoal or bar outside the entrance rather than by action of tidal flow.[3,4] With these figures in mind the inlet must

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[a] September 1960, by E. A. Hansen (Proc. Paper 2601).

[2] Head, Coastal Engrg. Lab., Univ. of Florida, Gainesville, Fla.

[3] Stability of Coastal Inlets, by P. Bruun and F. Gerritsen, North Holland Publishing Co., 1960.

[4] "Natural By-Passing of Sand at Coastal Inlets," by P. Bruun and F. Gerritsen, W. W. 4, December, 1959.

be classified rather as a nonscouring channel (of San Diego type) or as a harbor entrance than as a tidal inlet. Mr. Hansen, in the name of justice, clearly states that the tetrapod design had no responsibility whatsoever for the mishap. A special block very effective for energy destruction and very stable in a rubble mound was used in a misunderstood way for an inadequate design.

The Federal project now under construction (1961) includes two jetties of almost equal length about 2,300 ft. The bay channel is 14 ft deep at M.L.W. but all other channel dimensions are the same as with the first project. Due to the increased cross-sectional area of the bay channel current velocities in the channel part of the entrance area may exceed one ft per sec. Although flushing ability thereby will be somewhat improved, the inlet currents cannot be expected to be able to keep the inlet free of deposits and strong onshore storms with 3 ft to 4 ft ocean tides may cause scouring currents as well as deposits in the bay channel. As mentioned by the author, the existing channel has been subject to a considerable shoaling. In the future scouring and meandering currents will be infrequent but regardless of this the channel may present a material trap. Although the lagoon is very shallow, waves from the NW and N will be a few feet high during strong winds blowing lengthwise in the open lagoon. The existing spoil banks along the bay channel bear witness of such wave activity. In as much as the bay channel between Port Mansfield and the Gulf is 10 miles long, it seems to be a large and uneconomic undertaking to protect the channel from shoaling (that amounted to 300,000 cu yd per yr in the critical 1957 to 1960 period). Nevertheless, it may prove desirable to consider future use of spoil from maintenance dredging for construction of (stabilized) spoil banks to protect the most sensitive parts of the channel from functioning as material traps. There is another place at which a stabilization of the inlet channel may prove desirable in the near future and this is the shoreline area between the main jetties, at which a couple of smaller jetties inside the outer jetties (telescope arrangement) may prove practical rather than leaving this area free to nature's pleasure. From Fig. 7 it is seen that the channel at that point is supposed to narrow its bottom width from 250 ft to 100 ft. This will cause some local littoral drift bayward that may prove to be of such nuisance that it will be best to check it with a couple of small (inexpensive) sand trapping jetties or groins. Similar arrangements will be found at various places on the North Sea Coast.

Mr. Hansen writes that "at the end of 10 years, following completion of the jetties, the littoral drift would have filled the south jetty and started moving around the end of the jetty. The materials would then either be moved shoreward and deposited in the navigation channel, be moved seaward and lost, or be moved north across the entrance where it would not affect the project." Inasmuch as the inlet flow characteristics will only have changed little with the new project by-passing by flow activity will be just as inadequate as with the original project. Regardless of the length of the jetties sand accumulation can be expected particularly just inside the extreme end of the jetties, preferably the south jetty. The assymetrical location of the inlet channel (closest to the North side) is an advantage in this respect. There will probably also be a tendency to formation of a bar in front of the inlet entrance. The material for such shoal will be derived partly from rip currents that will run out along the jetties and partly by shoreward material creep on the ocean floor caused by the assymetrical water movement in the shallow water waves. Such material will usually return to sea in rip currents. In this case, part of it may be re-

turned to sea by the inlet ebb current but this current is only weak. Its interference with the bottom creep may, therefore, rather result in a stunted bar formation close to the entrance than in a tendency to formation of a channel in the offshore bottom. How important this tendency will prove to be remains to be seen but from wave data compiled by the Beach Erosion Board for this part of the Gulf of Mexico it seems as deep water waves of 6 ft or more may occur at least 200 hr per yr and their effect will be as previously described. The situation is probably to some extent comparable to the situation at Shark River Inlet at New Jersey[5], that seems to be bothered by a drift quantity of about 265,000 cu yd per yr partly on its updrift (South) side which is provided with a 950 ft jetty and partly in its 20 ft navigation channel which has to be dredged regularly.

Therefore trouble with deposits may show up immediately after completion of the new project.

The author stated that "a relatively impervious and sand-tight jetty is required to prevent the passage of littoral drift." Comparing the jetties now under construction with the initial tetrapod jetties, one may get the impression that the requirement of sand-tightness certainly has been taken seriously this time. A sand grain in order to pass through from ocean to inlet through the jetties would have to take an average trip of about 100 ft through a rock jetty built up of various layers and with a good core with little voids. Most likely it would not take the trip or at least never succeed in passing through. The jetties designed seem to fulfill all requirements for a substantial design and it may even be asked: Is it necessary to make them that heavy? In this respect two factors should be considered. One is the apparent desirability of avoiding too concentrated loads on the rather soft bottom and the other is the general maintenance problem. Padre Island is not a place at which one can run out every day to supplement or correct; yet it may still be asked: would't a lighter design, for example, a 16 ft wide stone crib with a permanent trestle consisting of prestressed concrete king-piles spaced about 3 ft apart and provided with a sand-tight wall on one side (or perhaps with asphalt grouting) suffice, at least for the parts of the jetties inside the 10 ft to 12 ft depth contour. In such case a tetrapod rubble mound resting on plastic or nylon cloth material to avoid bottom scour may prove the most economical solution. Although the author says that it will be very difficult to recover the tetrapods already in, they may still serve as bottom layer or a kind of foot protection for the next layers. Such crib or "permanent trestle" design is usually less expensive than the comparable all-rubble mound jetty. It only holds 40% to 50% of the quantity necessary for an all-rock design. It is easy to maintain and because of the application of prestressed concrete and creosoted wood for the upper part it is very durable although not everlasting as a granite jetty. Such stone crib was preferred by the State Road Department of Florida at Longboat Pass, Florida.

The predicted influence of the inlet on the material balance on Padre Island makes a Floridian feel the cold water running down his neck, but being in Texas and not in Florida and the expected shoreline recession on the downdrift (north) side that in its peak apparently is estimated to be of the order up to 50 ft per yr will not have similar real estate consequences as they would in Florida

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5 Shark River Inlet Sand By-Passing Project by W. Mack Angas, WW 3, September, 1960.

where it is unlikely that the State government would ever approve a Padre Island Inlet without an obligation of full by-passing. Because of the relatively unimportance (with reference to property value) of Padre Island no reason has been seen to offer such arrangement much thought at this time. But perhaps the inclusion of a "gate" (an opening) in the southern jetty leaving a future passage from the inlet to the accumulation area could have been considered. Such gate was recently suggested for the Boca Raton Inlet in the SE coast of Florida and is also under consideration for a new barrier inlet in West Florida. One thing seems to be important at Padre Island at this time and it is to let the wind blow all the sand it can from the ocean side to the bay side thereby moving the barrier bayward.

Summarizing the preceding remarks to Mr. Hansen's very detailed and thorough article, it may be said that a reader of the paper may get the impression that because too many errors were made initially, this time we will certainly do things well. The question is whether all the bugs are out. The answer probably is that most of the big ones are out but a few more may still remain. The writer regrets that in this country, contrary to the practice at almost any other place, there is a reluctance when it comes to model experiments with engineering projects on sandy coasts. Some engineers have the opinion this is not necessary, and enough about this is known beforehand, but they always seem to be representatives of the least practical experience.

The importance of model studies for design of inlet entrances and channels can hardly be overrestimated. Studies of flow distribution in the inlet will assure that the inlet, the bay channel, and the protective jetties get the best and most stable configuration. The influence of the jetties on the longshore drift can also be evaluated better in a model. Numerous experiments with tidal inlets and estuaries are at this time in progress in England, Holland, France, India, Portugal, and Denmark, as well as in various other places in the world, but only very few in the United States. At the University of Florida, one study was completed at Gainesville, Florida in 1960 but three studies will be completed in 1961 all financed by private and local government sources. Practical experience and experiments have proven that it is a misassumption that the inlet channel and its protective jetties shall always be straighlined. This problem can also be looked into by model experiments the cost of which are so low (usually less than 1 per mile of the total construction cost) that there is hardly any justification for omitting them. The nexessary funds should be provided for a more thorough planning.

DESIGN OF INLETS FOR TEXAS COASTAL FISHERIES[a]

Discussion by F. Gerritsen

F. GERRITSEN.[31]—The complex nature of the problem attacked by the authors is evident if the following features are considered:

1. Tides in the area are of the mixed, diurnal type with a wide variety in tidal range.
2. Each of the proposed passes, after construction, will cause a certain change of the tide characteristics in the bay.
3. Due to variations in run-off the salinity of the bay waters, according to the authors, varies from 1 ppt to 100 parts per thousand (ppt), in which the average salinity of the Gulf is about 35 ppt.
4. The area under consideration is occasionally hit by severe hurricanes, that will adversely affect stability conditions in the new inlets.

For a realistic approach the authors had to make a number of assumptions and simplifications.

The use of the hourly tidal differentials across the barrier island seems to offer a useful tool as a first approximation to analyze the tidal currents through the inlets.

For the computation of the potential tidal area and water exchange between the Gulf and the bay, the authors made use of some empirical formulas,[3] but the use of those empirical relations should be limited to the case in which the tidal area used in the formula is the actual area of influence (up to the tidal division line) of each individual inlet.

As in the design procedure inlet velocities have been determined by use of the tidal differentials, the tidal prism could more reliably have been computed from the velocities in the inlet directly.

In order to arrive at a design velocity for the proposed inlets, the authors have used Shields' analysis for critical tractive force representing that value of the boundary shear between water and bottom (τ_c) at which the force of the flow on the particle just overcomes its resistance to motion.[6]. Meanwhile, the use of this value as a basis for design is actually tied to the assumptions of "no sand transport" in the inlet channel. This can be seen from bed-load formulas, for example, as developed by Shields and others[6] in which the bed-load transport is expressed as a function of $(\tau - \tau_c)$.

The use of this criterion is possibly justified in the design of self-cleansing sewers, as referred to by the authors, but for the design of tidal inlets it is most likely to give values for the inlet current that are too low to give stable conditions.

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[a] September 1960, by H. C. Innis and H. P. Carothers (Proc. Paper 2603).
[31] Assoc. Prof., Coastal Engrg. Lab., Univ. of Florida, Gainesville, Fla.

Reference is made to a recent publication[32] on this subject, in which it is shown that stable inlet conditions for $\tau = \tau_c$ can only be obtained if the supply of littoral drift to the inlet (ΔM) is zero. In case ΔM—the intercepted littotal drift—has a certain value inlet stability requires $\tau > \tau_c$.

If a design of an inlet is based on a too low value of the design velocity, it may be expected to shoal simply because it will present a sand trap.

In the design procedure followed by the authors, the actual velocities will be somewhat larger than the values obtained in the first oreder computations. The tidal differentials tend to increase under the influence of the newly constructed inlet(s) so that this effect will counterbalance to a certain degree the effects of a low value of the design velocity.

The authors have recognized the importance of a Gulf bar as a regulating mechanism to the stability of a tidal inlet.

For a better understanding of the mechanism of transport, the following remarks may be helpful:

The dynamic balance is first of all governed by the total quantities of material transported through a cross-section during a certain time, in both the ebb and the flood direction.

If N_g indicates the total rate of material transport through the gorge, then the value of the integral $\int_0^t N_g \, dt$ (positive or negative) determines whether there is a predominant transport in ebb or flood direction.

If, in a similar way, N_b represents the total transport across the Gulf bar per unit of time whereby the positive direction coincides with the ebb flow, the requirement of a stable inlet between the gorge and the Gulf bar may be expressed by the relation:[32]

$$\int_0^t N_b \, dt = \int_0^t N_g \, dt + \Delta M_t \quad \ldots\ldots\ldots\ldots (11)$$

in which ΔM_t is the amount of material intercepted from the littoral drift during the time t.

The value of $\int_0^t N_b \, dt$, the total amount of material that is transported across the bar, consequently depends on the value of $\int_0^t N_g \, dt$ and ΔM_t for stable conditions.

In order to determine the values of $\int_0^t N_b \, dt$ and $\int_0^t N_g \, dt$, the velocity-time curve and the transport curve can be determined by tidal computations and computations on the rate of transport.

In most cases, however, a combination of currents and wave action is responsible for the transporting mechanism.

Unfortunately, theoretical knowledge on the transport characteristics under the combined influence of waves and currents is still very limited so that usually accurate results cannot be obtained by computation only.

For this reason the writer is in full agreement with the conclusions of the authors that for the final design of an inlet, a model study should always be considered in order to arrive at a higher degree of accuracy in the evaluation of the various parameters.

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[32] "Stability of Coastal Inlets," by P. Bruun and F. Gerritsen, North Holland Publishing Co., Amsterdam, Holland, 1960.

In this particular case the various proposed inlets could be considered in one model study whereby the boundary conditions in the model are adjusted for the characteristics of each particular inlet.

RUSSIAN RIVER CHANNEL WORKS[a]

Discussion by R. Robinson Rowe

R. ROBINSON ROWE,[2] F. ASCE.—The low value of riparian improvements in rural areas seldom warrants the high cost of positive training works, but often justifies modest investment in lighter and cheaper types of bank protection if there is a fair chance of success, or if bank recession or meander is partially abated. The author's account of this project is notable for describing adaptations of cheap types of protection to a suitable reach of Russian River and is especially commendable for reporting partial failures and corrective measures.

Failures are better teachers than successes in this field. Properly analyzed, each failure discloses something of the nature of the stream and why a device or layout could not cope with the power of the stream. The rapidity or finality of failure measures the deficiency of a first trial and guides economical redesign for a second trial. On the other hand, successes may have been very expensive and wasteful, with no measure of the premium paid for overdesign.

The account of failure of barrier jacks recalls observations by the writer of distress and failure of cable-connected jacks and tetrahedrons installed by the California Division of Highways. The cables had been designed empirically, guided by past performance and relative magnitudes of velocity and drift. On loss, an attempt was made to diagnose the deficiency as weakness of cables or vulnerability of anchorages. When distressed, a cable weakness would show in yielding of connections and tautness of cables. Many cables were so taut that they would hum when struck with a hammer. Such tension could not be fully explained by hydraulic forces impressed on drift-laden jacks and tetrahedrons.

In many such cases the tension could be ascribed to forces normal to the cable. In a simplified example, consider three cable-connected jacks in a string along the bank. Initially set on the bed, they have become firmly embedded by subsidence or depositon. Now let the middle one be so undercut by local scour that its outer legs subside, rotating the jack outward—a common occurrence. The incremental tension in the cables connecting this jack to its neighbors may be 10 times, or even 100, times the normal force. One solution for this hazard is to attach anchor cables low on the back of the string near the probable axis of rotation and to slacken all cables and fencing on the front of the string.

The author's barrier jacks were laid straight across the channel and anchored at each end, so that normal hydraulic forces on the jacks were greatly

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[a] November 1960, by Israel H. Steinberg (Proc. Paper 2647).

[2] Principal Bridge Engr. (Retired). Calif. Div. Hwys., Sacramento, Calif.

magnified at the anchorages. He rebuilt on a curve that reduced the magnification to 1.45 but added 10% to the length of the line, with apparent success. A different solution was used with tetrahedrons strung across the Colorado River near Blythe in 1926. The string was kept straight but cabled to concrete drag anchors cast in the bed directly upstream.[3]

In citing that "the use of jacks for river stabilization is not particularly unique," the author need not have gone so far as the Rio Grande for his example. At least 30 California streams could have been mentioned, notably the Eel, Pajaro, Salinas, San Benito, Santa Clara, Tule, and Van Dusen Rivers. Many of these are illustrated in a recent publication[4] that discusses their effect on channels and, in a general way, the forces acting on them, with conclusions on their adaptations to retards and jetties.

By not similarly citing earlier use of flexible fence, the author may mislead some readers to infer originality of this type. The flexible fence was developed[5] in 1941 by H. E. Rowe and Walter Gerow for use by the Spreckels Sugar Company at several locations along the Salinas River, where it has served surprisingly well for 20 yr in spite of its low cost and light-weight construction. However the author must be commended for successfully transplanting the idea to a stream with much heavier drift and bed materials.

OTTO C. VON SEGGERN,[6] F. ASCE.—The paper is essentially a favorable 4-yr progress report on the results of investigations, observations, and studies over the test period 1956 to 1960 on performance of channel works at demonstration sites in the several problem areas in which selected types of channel stabilization works were constructed and maintained in a 4-mile reach of the Russian River above Geyserville.

The writer worked from 1944 to 1946 and also before on investigations for possible channel works on the Salinas and Pajaro Rivers (California) and then on the preparation of the original survey report during its formulative planning stages including plans for channel stabilization works for the authorized project on the Russian River.

The need was recognized for the provision of a well organized program of initial construction, maintenance, observation of performance tests, and progress reports at selected demonstration areas. This would be necessary in order to demonstrate the effectiveness of the plans for the proposed channel works. Performance under flood conditions would dictate the remaining channel stabilization works to be constructed. This is essentially what the district engineer recommended in the survey report.[7]

It is almost certain that without this provision for initial construction and continued tests, the channel stabilization feature would not have been included in the project. However, neither would it have been practicable to have conducted such initial construction and tests during the preparation of the survey report; although the need was recognized at the time. Therefore, in order to

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[3] "River Control in the Palo Verde Valley," by Spencer E. Webb, Engineering News-Record, August 11, 1927.

[4] "Bank and Shore Protection in California Highway Practice," by N. R. Bangert, R. M. Carmany, R. R. Rowe, and P. R. Watson Jr., Calif. Div. of Hwys., 1960.

[5] "Steel Rails for Bank Protection on Salinas River, California," by H. F. Cozzens, Civil Engineering, Vol. 16, p. 114.

[6] Senior Engr. W. S. Major Project Sect., Resources Planning, Calif. Dept. of Water Resources, Sacramento, Calif.

[7] "Russian River, California," House Document 585, 81st Cong., 2nd Session, May 17, 1950.

expedite the preparation of plans for channel protection on the Russian River, prior to studying test sections, consideration was given to other channel works developed on earlier investigations by observations of performance of existing works on the Salinas River from 1939 to 1946, as well as the Pajaro River before 1941. The types of channel works considered most suitable for the Russian River conditions were selected. Minor modifications were made for the survey report, with the provision that their effectiveness would be studied under test conditions on the Russian River prior to being adapted for the entire project.

There was some degree of risk that the works as proposed in the survey report for the Russian River would fail to perform as well as the channel works on the other two rivers and that extensive modifications of the Russian River plans might be necessary.

It was with these conclusions in mind that the author and other members of the San Francisco Engineer District selected some of the most difficult problem areas on the Russian River and constructed several types of channel works as shown in the paper. Necessary observations and tests were then performed as authorized for demonstration purposes.

However, the risk of failure to perform satisfactorily is now practically removed, because in the report of the last 4 yr of observations and tests, the results are favorable. This demonstrates again that the principle of using this type of remedial works on a meandering river channel is correct. Well developed plans may be transferred to other sites within reasonable limits, because the works designed and constructed on the Salinas River performed substantially as planned when constructed on the Russian River. It is reported, that with some minor modification of details, the test site plans would serve as the basis for the design of project construction plans for the remaining part of the 100 miles of channel stabilization works on the Russian River.

Apparently, the structural failure of the "Barrier Jacks," as described in the paper and illustrated by Fig. 10 and Fig. 11, was due to a weak structural detail that has been corrected in the revised designs. This case also demonstrates that the forces that develop in the cable-channel type of channel work must be recognized and properly provided for in order to avoid failure. The "Barrier Jacks" were used to form a permeable dike or dam, and as a result at flood stage, the pressure against them would be similar to the pressure against any other type of dam.

Considerable information on the performance of channel works at several demonstration sites and the loading developed on the cable-anchored types of channel works during flood stages were developed during the earlier investigation on the Salinas River. Parts of these Salinas River data studies are presented elsewhere.[8]

The initial channel works investigations were started in 1939 on the 90 miles of main valley of the Salinas River from the mouth upstream to above San Ardo. These studies were conducted primarily to observe the performance of the several parts of the channel and bank protection works that had survived the 1939 floods.

During the time when the initial channel works investigations were started in 1939, H. F. Cozzens, F. ASCE, was the County Engineer and Road Commissioner of Monterey County. He had previous experience as the engineer for a

[8] Unpublished Appendix III, "Field Surveys Observations of Bank Protection and Channel Training Works and Special Investigations" to House Document 208, 80th Cong., 1st Session, 1948, "Interim Flood Control Survey Report, Salinas River, California.

large part of the earlier bank and channel works that had been installed on the Salinas and Pajaro Rivers and other county streams dating back to 1910.

Mr. Cozzens has furnished information on the construction, maintenance, and performance of most of the bank and channel works installed prior to the 1914 and 1938 floods. The bank and channel works prior to the 1914 flood consisted mainly of the permeable wooden pile jetty type. It has been reported that a large percentage of these jetties was washed out and lost during the 1914 flood. Shortly after this devastating flood the first line of "jacks" was installed on the Salinas River, with very effective results. A few years later the State Highway Department prepared plans for installing "tetrahedrans" with 30 ft steel rails. These were installed at the King City and Soledad Bridges and both projects performed very satisfactorily. During the period after the 1914 flood and before the 1938 flood many of the riparian owners on the Salinas and other rivers in the county, installed bank and channel works at their expense, under the supervision of Mr. Cozzens as their engineer. About 20 projects were installed which varied in length from 500 ft to 1.5 miles. A substantial part of these works survived the 1938 flood.

It was these surviving parts on which the initial channel works investigations were started in 1939. These works had been designed and constructed earlier at the expense of the riparian owners. The performance of these works provided an excellent opportunity to make the needed observations and study of channel works. It is reported the damage to these earlier works and to bank cutting or erosion amounted to nearly $300,000 during the 1938 flood that had a peak discharge of 75,000 sec-ft at the gage near Spreckels. The river channel within the main valley is wide and shallow with wide sand and gravel bars, poorly defined, generally unstable banks, and having a capacity that varies from 20,000 cfs to 42,000 cfs. Over-bank flooding occurs throughout the reach. The river meanders and much damage is due to loss of valuable crop land from bank cutting and erosion.

The information in the first set of observation reports was used to initate correspondence with the offices of Chief of Engineers and the Waterways Experiment Station, with a view to possibly making some simulated scale model tests in the hydraulic laboratories. This would expedite the observations of performance of the several most promising types of channel training and bank protection works under study at the time.

However, after the details were furnished on the stream regimen and the peculiar characteristic of the river to carry large bed loads of material at relatively low velocities in its meandering channels, it was concluded that simulated model tests could not be made with any assurance of producing reliable findings. Therefore, a program of continued observation of the full-scale works at the selected sites over several seasons including above normal floods would be the only procedure that could be relied on for good test results.

One of the most outstanding demonstration sites is located on the right bank of the Salinas River about 4 miles below King City near river mile 62.6. According to the river surveys, a strip of valuable sugar beet land, that averages about 800 ft wide and nearly 10,000 ft long located between river mile 62.6 and 60.8 on the outside of the river bend, have been gradually lost to bank erosion and the meandering river channel during the 30 yr since the first channel survey in 1912.

In 1941 and 1942, plans for channel stabilization at this site and at other points on the river to prevent further damages were developed jointly by the

engineers in the San Francisco Engineer District and the local engineers on flood control for Monterey County and the Spreckels Sugar Company. Engineers from the State Engineers Office, Division of Highways, and the Soil Conservation Services participated in the planning.

At the King City site, plans were developed for construction of the anchored steel jacks for bank protection and for a system of cable-anchored floating training works. The plans were similar to those shown in Fig. 2. The channel stabilization works at the King City site were constructed by the sugar company at their expense during 1942 and 1943. They were essentially in accordance with the joint plans except large timbers were used to frame the jacks.

Since then the works have been maintained, including the construction of pilot channels across the higher gravel bar area, by the sugar company. The upstream end of the eroding right back channel line was actually pushed back from 800 ft to 900 ft to the original 1912 channel position, during the first high water stage in 1944 shortly after the channel training works were constructed. Subsequently, the upper half of the right bank channel line has progressively shifted back to points paralleling the "bent chord" alignment of the channel training works that were located near the old 1912 bank line. From this point, the channel extends downstream towards the anchor end at river mile 60.8. The bay area formed by the 1944 bank line and presently established bank line along the "bent chord" of the cable anchorage of the channel training works has been filled in by extensive silt deposits and supports a scattered growth of willows and brush.

It has been observed that after the channel training works are installed, an ordinary meandering river will follow the "bent chord" lines of the training works and begin to establish a more natural well stabilized bank line. The only vulnerable spots are the cable-anchor zones, in particular for the main line cables, because the alignment of the "bent chord" cable is held in place by the anchors at each end and by the intermediate cable-anchored lines across the bay.

The question of loading on the cable-anchored systems was a structural problem which was solved by a series of loading tests on a full-scale jack loaded with a natural accumulation of river driftwood. This jack was tied to a cable and towed in still water at various speeds to represent the different velocities of normal stream flow from 2 fps to 7 fps. The results are shown on a figure presented elsewhere.[9]

The plotting points from the curve are:

Velocity, in fps	Tension in cable to jack, in lb
2	125
3	300
4	575
5	875
6	1,200
7	1,500

These loads were used to estimate the loading that would accumulate from a series of jacks in a cable-anchored channel training works under an initial loading based on 1938 flood conditions.

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[9] Ibid., Fig. 3.

These loading tests also showed the need for providing structurally competent anchors for the elements in any type of bank protection or channel training works system. Unless such anchors are provided, considerable damage occurs to such works during flood stages when stream velocities and corresponding loads could exceed the range covered by the tests. It has been demonstrated that the effectiveness of the several types of bank protection and channel training works may be substantially increased by the construction of structurally competent anchor systems. Such systems may consist essentially of either: (1) plans for bank anchored cables to which the jacks or other elements are securely attached, or (2) a plan for which the anchors are deeply buried in the stream channel below the river "bed flow" depths. Loading tests were conducted on a structural scale model of the channel training works for the King City demonstration site. These tests showed that the upstream anchor points of the cable system at river mile 62.6 would be subjected to a maximum tension load of 100 tons during the initial loading from a flood of the 1938 magnitude. A peak discharge of 75,000 sec.-ft was reported on the Salinas River near Spreckels in February, 1938. These tests also showed that the intermediate cable anchor lines would be subjected to a maximum tension load of 33 tons during the initial loading.

One of the other outstanding demonstration sites is located at river mile 15.4, a short distance above the sugar mill at Spreckels. At this point, on the right bank of the river a critical bank erosion condition had developed in a soft silty soil. Erosion had caused a short oxbow-type river bend that was moving progressively downstream towards the sugar mill area by 1942. Something had to be done to stop this bank erosion so the engineer for the sugar company, H. E. Rowe, designed and supervised the construction of a section of the cable anchored "flexible fence," type of bank protection works. This is practically the same as shown by Section A-A, in Fig. 2. After the flood of 1944, the bottom of the fence was buried apparently to the depth of the river "bed flow" reached at flood stage that, in fact, prevented the river currents from undercutting the works. The bank erosion was stopped at the fence line. The works were rebuilt by splicing out the post rails, moving the supporting outriggers back to their original position, and adding more anchor cables and wire mesh. Since then a heavy volunteer growth of willows and brush has been established along the bank line and in the fence sections and apparently the danger to the sugar mill has been removed. This bank protection was accomplished without the use of channel training works to divert the main flood flow back to the original channels.

Another of the notable demonstration sites is located on the left bank of the river about 1/4 mile upstream from the Spreckels Bridge. The County of Monterey constructed channel training works and a flexible-type bank protection fence at this site during January, 1945, to protect the nearby county road from damage due to continued bank cutting. On February 3, 1945, the river reached its flood peak for the season and was overflowing its banks. From observations made during the flood stages, it was concluded that the floating type of channel training works were very effective at this site in diverting the flows back into the regular channel alignment. In the intervening period no more land has been lost in this area, flows have been channelized, and the banks have been stabilized.

Another of the important demonstration sites is located on the right bank directly upstream from the Salinas River Highway Bridge at King City. These

works consist of a long line of heavy cross-braced cable-anchored steel tetrahedrons. These works were constructed during the summer of 1938 to protect the state highway bridge after much of the existing upstream bank works had been lost during the February, 1938 flood. The river currents were endangering the bridge approach by severely eroding and undercutting the bank immediately upstream from the bridge. Numerous observations were made on the performance of these works and by May, 1944 or about 6 1/2 yr later, a full grown thicket of volunteer willows, cottonwoods, and other vegetation had taken root and firmly established itself around the steel frames with the wire mesh. This has resulted in the deposit of 7 ft to 10 ft of river silt in the long strip of new land between the new channel bank paralleling the front line of the tetrahedrons and back to the base of the old eroded bank line of 1938.

The performance at many other selected sites was observed during the 6-yr observation period from 1939 to 1945. The studies included several types: particularly, permeable jetties, flexible bank fence, wire mesh revetment, steel tetrahedrons, jacks, and the use of these types in several cable-anchored systems, including the large wooden floating type cable-anchored jack.

The Salinas River is a meandering stream and at flood stage carries a heavy load of light sandy silt and floating debris. All of the preceding types are more or less effective in collecting the floating debris and thereby assist in retarding stream flow along the bank lines, and thus resulting in depositing more silt in this area. During the growing season, the debris takes root in the new silt deposits. This results in a structurally-anchored root growth that increases the effectiveness of the works, unless undercut or flanked by further bank cutting at the ends. The failures of these systems have been due principally to their lack of continuity in the problem reaches or by loss to undercutting. These failures were substantially reduced where well designed anchor cable systems had been constructed, and properly maintained. It was concluded from the series of observations and studies of the existing works and their performance at the several demonstration sites on the Salinas River that it would be possible to design, construct, and maintain a feasible channel improvement plan. The studies indicated that more attention should be given to the possibilities of using the channel training works for a preventive or stabilizing purpose. This would gradually accomplish a desirable change from the presently meandering river channel into a well-aligned stable channel system. These works would naturally reduce the tendency for the formation of the highly erosive short "oxbow" type of river bends. When these conditions develop, the construction of extensive bank protection works are required to stop the damages.

This discussion is presented in order to give some information on the problems and the indeterminates that surround the planning and design required for fully effective bank protection and channel training works on an ordinary meandering river channel. It is believed that Mr. Steinberg has also demonstrated how much care must be used in the testing and design for this type of specialized channel works. He has particularly pointed out the need for utilization of well tested plans in designing and building flood protection works. It should be apparent that the plans for construction of extensive channel works on a river should always include a program of supporting tests at demonstration sites at the most difficult problem areas to assure insofar as possible satisfactory project performance. Considerable time is needed in

the planning stages in order to accomplish these tests so as to include above normal flood stages, to properly stress the remedial works.

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SACRAMENTO RIVER DEEP WATER CHANNEL: FUNCTIONAL PLANNING[a]

Discussion by Blair T. Bower

BLAIR T. BOWER,[2] M. ASCE.—Two points should be raised about the benefit computations in Mr. Gomez's paper. The first relates to the unit benefit figure for deep draft traffic. The second relates to the method of computing project navigation benefits.

With respect to the unit benefit figure for deep draft traffic, the "cheapest alternative means" for transporting the same commodities is not specified, but the cost of this alternative is given as $3.17 per ton. It is not specified but presumably this represents the weighted average of the rates charged the used or shipper, rather than the actual costs of shipping by the alternative means. From the standpoint of society this is not the true alternative cost. From society's point of view, it is only the difference between the costs of transport by water and the long-run out-of-pocket rail or truck costs that should be counted as the benefit for the waterway project. This is because the difference between rail or truck rates and out-of-pocket costs represents the portion of the system overhead that is contributed by the particular commodity. When the commodity is diverted to the waterway, system overhead obviously continues and the railroad or trucking firm either suffers a loss of income or the rates on other products shipped by rail or truck must be increased to cover the loss. Thus, from society's standpoint the unit net benefit would be the difference between the waterway cost and the long-run out-of-pocket cost of the alternative means of transport. The typical gap between rail rates and out-of-pocket costs is on the order of 22-30%, according to Otto Eckstein.[3] Assuming this gap to be 25% in the Sacramento Channel case, for the purpose of illustration, then the unit benefit for deep draft cargo transport is $1.61 per ton instead of $2.40 per ton.

The second point relates to the method of computing total project benefits from navigation. Future traffic has been estimated over the 50 yr period under consideration. The benefits in any year of this period are simply the number of tons carried times the unit benefit per ton. However, in deriving the average annual benefit for the project, the average traffic for the total period times the unit benefit is used. This implies that a dollar of benefit has the same value whether it accrues in the first year or in the forty-fifth year. This in essence is assuming that $1.00 forty-five yr hence is equal to a $1.00 in the present time, that is assuming an interest rate equal to zero.

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[a] November 1960, by Amalio Gomez (Proc. Paper 2649).

[2] Asst. Dir., Meramec Basin Research Project, Washington Univ., St. Louis, Mo.

[3] "Evaluation of Federal Expenditures for Water Resource Projects," by Otto Eckstein, U. S. Congress Joint Economic Committee, Federal Expenditure Policy for Economic Growth and Stability, 85th Congr. 1st Session, November, 1957.

However, zero interest rates generally do not exist in the real world. Future returns from an investment are discounted by some interest rate.

If the benefits for each year were discounted back to the present using some discount factor, the present value of these benefits would be determined. The present value could then be compared with the total of the first costs in order to provide an indication of the worth of the project. The present value of the navigation benefits has been computed by the writer. The assumptions involved are as follows: (1) the unit benefit for deep draft cargo is $1.61 per ton as derived previously; (2) the traffic in any given future year, both for deep draft and shallow draft, was determined from the data given by Mr. Gomez, assuming a straight line variation between the given points; (3) the unit benefit figure for shallow draft cargo is $0.125 per ton, as given by Mr. Gomez; and (4) the interest rate is 2 1/2%, in accord with general Corps of Engineer practice.

The present value of the gross navigation benefits for the 50 yr period with these assumptions is about $51,300,000. This contrasts with the $128,000,000 implied in the data given by Mr. Gomez, that is, $2,560,000 per yr times 50 yr. Because no annual operation and maintenance costs were included, comparison of the present value of benefits (benefits in each year minus the operation and maintenance costs for the year, discounted), with the total first costs is not possible. However, the large difference between discounted benefits and undiscounted benefits suggests that the project may well be of dubious economic justification, even with the low 2 1/2% interest rate. Even using the $2.40 per ton unit benefit figure, the present value of the navigation benefits for the 50 yr period is only about $70,300,000.

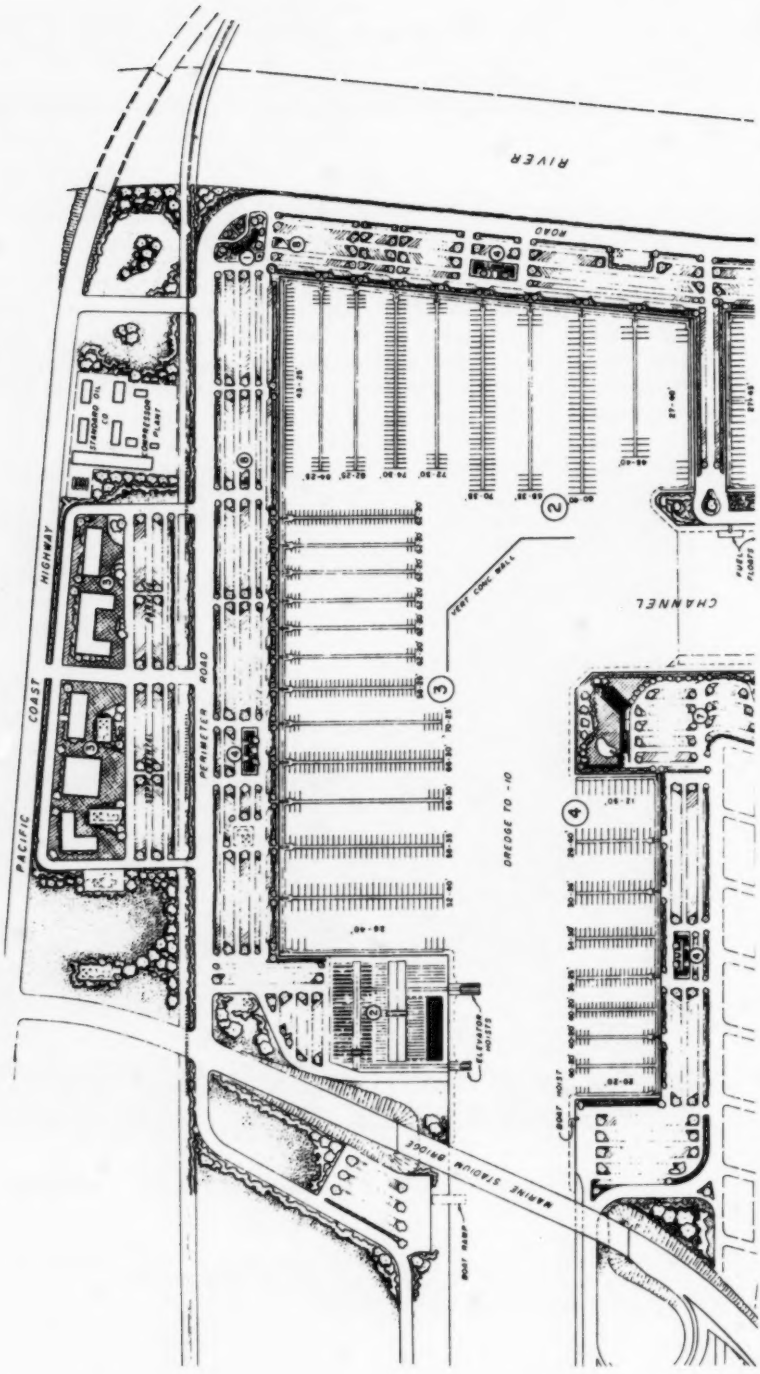
DESIGN CONSIDERATION FOR CALIFORNIA MARINAS[a]

Discussion by J. W. B. Blackman

J. W. B. BLACKMAN,[19] F. ASCE.—The enormous growth of the pleasure boat industry is indicated by statistics of expenditures for 1960. It is estimated that $2,000,000 was spent on marinas, mooring facilities, boats, motors, repairs, fuel, trailers, and club houses. It is expected that a similar amount will be spent in 1961.

FIG. 11.—LOS ALAMITOS MARINA, LONG BEACH, CALIFORNIA, 1960.

[a] November 1960, by James W. Dunham (Proc. Paper 2658).
[19] Cons. Engr., Long Beach, Calif.


RIVER
ROAD
CHANNEL
PACIFIC COAST HIGHWAY
PERIMETER ROAD
DREDGE TO -10
MARINE STADIUM BRIDGE
ELEVATOR HOISTS
BOAT HOIST
BOAT RAMP
PUMP FLOOR
STANDARD OIL
COMPRESSOR PLANT

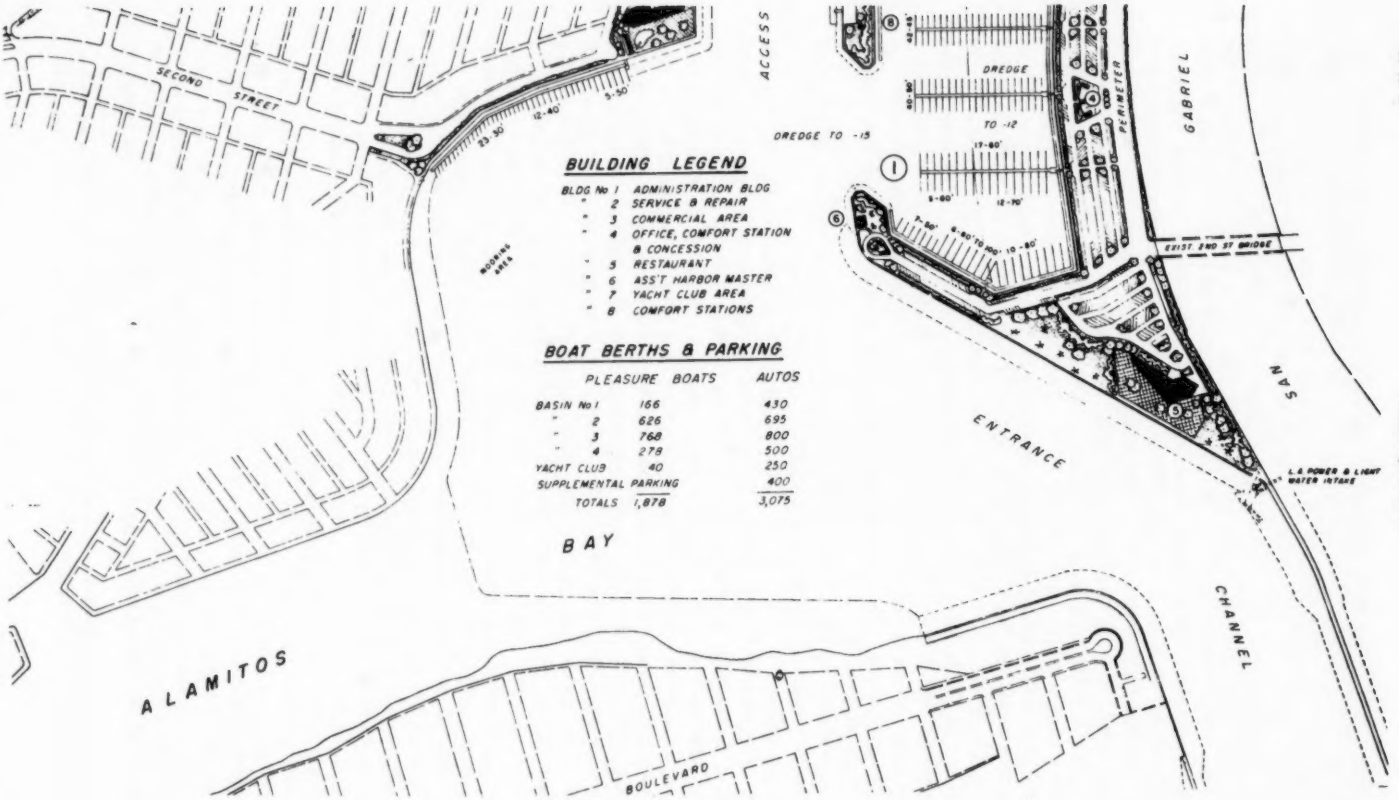


FIG. 12.—ALAMITOS BAY MARINA

Of concern to engineers designing harbors on the Coast of California are the unpredictable tidal waves caused mainly by earthquakes on the West coast of South America. The effects of phenomenal wave actions cannot be overlooked. The earliest Seiche of any magnitude was noted in Los Angeles Harbor in 1877. It was given in a report of the Chief of Engineers of the U. S. Army that stated, "about 7:00 a. m. on May 10, the tide being about 4 feet above low water, the sea began to rise and in 25 minutes rose to 6.8 feet in twenty five minutes." Several others have been recorded from time to time, and have been described by Harry Leypolt.[20] For engineers contemplating designs of harbors of the California Coast, the prior referenced papers are well worth studying, and as the author suggests, specialists should be consulted.

On May 23, 1960, following the Chilean earthquake, the surface of the water in Long Beach Marina rose to 3.8 ft to 4.1 ft. (tide guage at Marina) One of these surges occurred at 4:54 P.M. Had this particular surge happened 3 hr later at the predicted high tide of 5.7 ft, it would have raised the surface of the water to El. + 9.8. Low lying land in a residential area in the vicinity of the Marina would have been flooded. Fortunately no damage was done to the Marina or to any of the craft berthed therein. Considerable damage was done to many small craft in the Cerritos Channel, that joins Long Beach Harbor to Los Angeles Harbor. Two hundred and forty boats sought refuge in the Marina upon the suggestion of the U. S. Coast Guard.

Not mentioned by the author is the importance, in some cases, of studying a prospective Marina by building a model.

Before design studies were commenced on the Long Beach Marina, the writer, recommended that funds be provided to build a model. A model was built to a horizontal scale factor of 300 to 1. The vertical factor of 30 was selected, that gave a ratio of horizontal to vertical scales of 10. The operation of the model revealed interesting situations. The overall currents in the entrance channel produced by tidal changes were substantially reduced by the combined effect of widening and deepening; with the increased water area within Alamitos Bay, the result of this was a maximum of 0.62 knots, that has had little or no effect on small boat navigation. Protection from waves generated within the bay area was materially assisted by the building of a baffle wall, facing the water with the maximum fetch. This was the most effective protection for Basins Nos. 2 and 3, Figs. 11 and 12. The velocities at the entrance channel, and the center of the access channel adjacent to the basins, and the movement within the basins, indicate a satisfactory change of water. It is estimated that nearly $2,000,000 was saved in the cost of building the Marina by using data compiled from the model studies, as compared with tentative layout plans suggested by other agencies. Also, 400,000 additional sq ft were made available for mooring facilities, increasing the revenue from boat rentals above what was anticipated.

As the studies proceeded, the shapes of the basins were selected after different shapes had been discarded by a process of trial and error. The studies revealed that a satisfactory plan could be made to protect small craft from excessive wave action.

Because the recently completed Long Beach Marina is now in full operation, with 1878 boats moored within its basins, a general description would be of interest, as it amplifies successful design practices contained in the author's paper.

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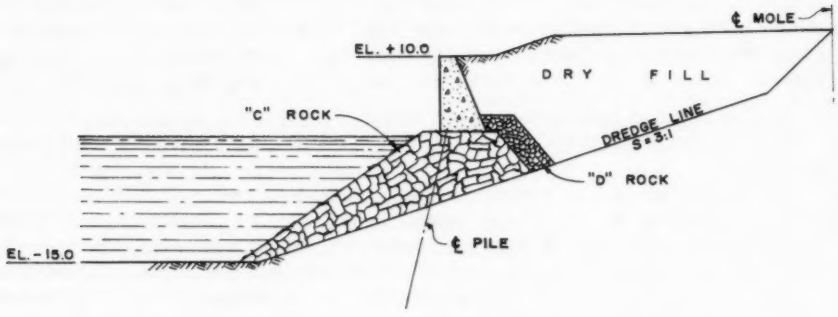
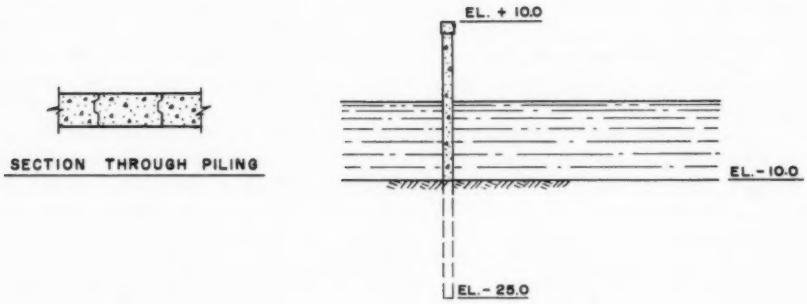
[20] Proceedings, U.S. Naval Inst., Annapolis, Md., Vol. 63, No. 6, No. 412, June, 1937.

The entrance channel from the Pacific Ocean to Alamitos Bay is 3300 ft in length, and lies between two rock mound jetties. The bottom width is between 459 ft to 500 ft. Dredging from the Ocean landward, and finally connecting with the Marina Stadium: the first 2000 ft was dredged to - 20 (the level of the ocean floor at this point), thence to - 15 to basin No. 1, and to - 12 and - 10 in the remaining basins. The perimeter of the basins is lined with a gravity wall, Fig. 13 (with no steel reinforcement) built on piles and a rock embank-



(A)

MASS CONCRETE BULKHEAD



(B)

SHEET PILE WAVE BARRIER

FIG. 13.—GRAVITY WALL

ment, the toe of which terminates at - 15, and - 12. The writer does not favor the use of steel sheet piling for bulkhead walls mentioned by the author. The preventative measures to arrest oxidization are, in his opinion, too costly.

To prevent deterioration from exposure to sea water and sea air, every endeavor was made to utilize the latest materials and methods available that were resistant to disintegration or oxidation.

Larger boats are berthed in Basin No. 1 near the entrance channel. Launching ramps for portable craft are located on The Marine Stadium, some distance from the moored boats. Two fuel barges are berthed in the main channel, a safe distance from the boat basins. A boat repair yard equipped with two mechanical hoists is conveniently located, and has a large area in the rear available for dry storage. Rest rooms are spaced at intervals along the perimeter wall. A Club House has been erected by the local Yacht Club. Electrical energy, telephone, and water are available. A fire station is situated in Basin 1 and fire hydrants are set at close intervals. A locker box is provided for every boat. Paved parking is provided for 2825 automobiles immediately behind the respective basins. Landscaping has added beauty to the Marina. Special features in connection with the design and its construction are:

1. The building and operation of a model.
2. The entire decking is floated on 6700 fiberglass-reinforced plastic pontoons.
3. The harbor is well protected from large waves.
4. The railing on top of the perimeter wall and all gangways are of 63ST 6. aluminum alloy, with 0.0015 in. anodized surfaced treatment. The top rail is 2 1/2 in. outside diameter and the middle rail is 2 in. outside diameter.

Financial.—The building of the Marina, the purchase of land, removal of five bridges, and of a railway track, dredging and the construction of jetties, were entirely financed from royalties received from the City of Long Beach oil wells. Therefore, there should be no expense to the taxpayers, when all the fees from the fully occupied basins are collected. The cost of the Marina alone was $8,000,000.

GREAT VOLGA WATERWAY[a]

Discussion by Roy F. Bessey

ROY F. BESSEY,[11] F. ASCE.—Mr. Kabelac has done American engineering a distinct and unusual service by presenting a view of the engineering development of the Great Volga Waterway over the past 35 years. In the writers' opinion it is desirable that more is known of Russian developments—both for an understanding of the breadth of their underlying socio-economic engineering concept and for their progressive and often striking engineering features.

The imaginative and bold approach of Russian planning was impressed upon the writer personally during a preliminary examination—begun in the USSR in 1930, but never carried through—of proposals. These proposals included Mariinsky, Volga, and Kama-Pechora projects, and outlines of larger plans of the "five-seas" and similar character. The writer has retained a general interest in the progress of such plans. Other sources of general information might be of interest.[12,13,14]

It is believed that something very substantial can be learned from a knowledge of Russian plans and projects, including the hydrologic inadequacies and other conditions or results, favorable or unfavorable, mentioned by the author. A greater technical exchange between the planners, engineers, and scientists of both countries, and in the United Nations at large, would be of mutual advantage. The paper by Zvonkov,[12] proposes a wide international exchange of experience and information, including surveys, basic data, hydrology, hydraulics, floods, channels, siltation, ice, reservoirs, hydroenergetics, hydrotechnics in land improvement, transport, water supply and protection, etc. Zvonkov also proposes plans for organizational institutions and measures to facilitiate such exchange. Divergent views should not stand in the way of careful and open-minded appraisal of the science and techniques of river-systems, water, and power development.

Russian river and harbor plans should be of interest to many Americans concerned with the development of river-system resources and not only engineers. The following points seem worthy of general and serious consideration.

Over-All River-System and Inter-System Scope of the Planning.—The "five-seas" concept, mentioned by Mr. Kabelac, with its projected waterway inter-

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[a] February 1961, by Otakar W. Kabelac (Proc. Paper 2749).

[1] Cons. Engr., Portland, Oreg.

[12] "Integrated Water Resources Development in the River Basins of the USSR," by V. V. Zvonkov, Academy of Sciences, USSR, Tech. Sciences Sect., 1957.

[13] Proceedings, Regional Tech. Conf. on Water Resource Development, 1956, UN Economic Commission for Asia and the Far East.

[14] Reports on Relative Water and Power Resource Development in the USSR and the USA, by Committees on Interior and Insular Affairs and Pub. Works, U. S. Senate, January, 1960.

connections from the heart of European Russia to the Baltic, White, Barents, Caspian, and Black Seas, is illustrative. Other impressive interconnections are outlined in short- or long-range plans. Examples are, The Dniepr connections to Volga and Baltic; the West-East interconnections of Volga, Ob, Yenesei, and Lena systems, through the large lateral tributaries, to serve the large resource-extraction and heavy-industry complexes in Siberian and European Russian regions.

Inter-System Hydrologic Research and Planning. —The general objective of diverting water from the northward-flowing Arctic reaches to temperate-zone streams and basins, where there is the greater prospect of beneficial use, has very deep meaning in Russia. In North America, the principle is most likely to have important meaning in connection with the Mackenzie and Yukon systems.

Mr. Kabelac mentions the Vychegda-Pechora-Kama transfer, through which part of the waters of the Northern Dvina and Pechora can be diverted southward to help fill deficiencies in the lower Volga and the Caspian Sea and to check regression in the latter's level. There is also a proposal for a long canal east of the Urals to relieve the shortages[12] apparently following upon the Volga development.

> "The lack of water resources in Central Asia and the dislocation of the water balance of the Caspian Sea have raised the question of shifting the waters of the Ob and even of the Yenesei Rivers, which now flow uselessly into the Kara Sea, into the Caspian Sea. Preliminary considerations envisage the construction of highhead dams on the Ob River to send the waters along the Irtysh and the Tabol Rivers via the interbasin canal (the Turgai Gates) to the Sea of Aral, and then through a canal to the Caspian Sea.[11]

The broad scope of inter-basin plans—including measures to avoid or mitigate serious dislocations or imbalances in hydrologic or ecological conditions in affected waters—was discussed in more popular vein during the earlier stages of planning and development.[15]

Strong Place of Inland Navigation in River-System Development and in the Russian Economy.—In regard to waterway development, it is significant to discuss further the expansion of the U.S.S.R.'s inland waterway lines and traffic. Between 1913 and 1955, their lines have more than doubled, from approximately 40,000 miles to approximately 82,000 miles. Traffic has not moved upward as rapidly, but the waterway improvement seems to be coming from a developmental into a pay-off stage, in view of the mileage increases of past and current years. According to available figures, the expected jump in ton-kilometers is from about 85 billion in 1958, to about 140 billion in 1965, and the extensions and improvements are to go forward on a large scale.

From rather limited knowledge, it would appear that their prospective rate of development is greater than our own. The Zvonkov paper[12] indicated about 86 billion ton-miles for 1960, which compares with a United States level of about 250 billion. However, over half of our total, or about 130 billion, is on the Great Lakes—a very special facility not paralleled in the USSR. Thus, outside of the Great Lakes, the traffic of the United States and the U.S.S.R. is of the same general order or magnitude, but that of the latter seems more

[15] Men and Mountains, by Ilia I. Marshak, 1935.

likely to increase strongly under the pressures of development of material resources and industry, and under the pull of a continuing waterway development.

Power Development.—From the writer's veiwpoint, there are strong challenges in the U.S.S.R. power development. Projects and plans encompass the large plants of the Great Volga waterways, but also the greater power streams of Siberia—the Ob, Yenesei, Lena, and Amur. Most notably, perhaps, the development of the strategic Angara, regulated by Lake Baikal, is well under way.

With this broad development, hydro-generating units and power plants exceeding any of our own are coming into being, and transmission lines of greater length, voltage, and capacity than any we have yet developed are appearing. Over-all rates of development of power capacity and output are increasing in considerable excess of ours, and, under present compounding rates, the U.S.S.R. seems likely to achieve parity within a score of years.[14] The challenges of our own growing need, pointed up perhaps by Russian achievements, would appear to the writer to point up desirable American policy and program directions, for example, the appraisal of all energy sources, the formulation of a progressive national power supply plan, and the acceleration of well-supported and correlated research and experimental work in power generation (including nuclear power) as well as energy transmission, interchange, and utilization.

Irrigation Development.—In the field of reclamation, it is also desirable to fill out the picture of the bold plans and projects of the U.S.S.R. for watering of the trans-Volga, lower Ural, Caspian, and Central Asian steppes.[12,14] A striking proposal in this connection is the Ob-Aral-Caspian canal, mentioned previously.

Of course, American geography does not lend itself to the development of plans of any high degree of similarity, except perhaps as to the vital elements of general boldness of concept, purpose, and execution and as to the close ties in broad multiple-purpose basin and inter-basin planning.

Stress on Multiple-Use and the Comprehensive, in Over-All Planning.—The author's paper notes the attention given to this aspect of development—and the correlation of development of navigation, power, flood control, irrigation, and other uses, the development of land and water transport, and the general land, water, industrial, and community development in the Volga-Don region.

The writer was impressed in the U.S.S.R., and later in India, with systematic efforts to consider full use of all waters of a river basin for all beneficial purposes, to lay out optimum plans of long-run use, and to develop specific and economically justifiable projects and programs within that unified and idealized general river-basin scheme. The logical concept is to see the whole, to get the best plan of control, to hold that plan together in its essentials, to refine and to improve as required from time to time, and to select from the plan feasible units for development.

In the United States, the approach, from the writer's viewpoint, tends to be angular and piecemeal. Too much energy is oriented to one or two purposes and to individual projects, and these projects often lack cohesion, and are, accordingly, vulnerable to attack and disruption in detail. The better mode is to get the complete basin plan, arrived at on the broad engineering and economic basis, to defend it as a whole, and to use economic analysis as a constructive tool to determine the best project features and priorities within the determined general plan.

This kind of approach has much more than ordinary meaning for the Rampart project and Yukon Basin planning—a great area where river-system development is beginning from the ground upward. Success in a relatively remote and deeply under-developed region is highly dependent upon simultaneous research and planning for coordinated and comprehensive action in engineering, industrial location, power transmission, transportation, community expansion, etc. The engineering project cannot stand alone in the wilderness. Its uses and supports must be developed with it.

Stress on Scientific Research and Development in River-Systems and Related Fields.—In the view of the writer, there are challenges in the attitudes of the Russians toward scientific and technical research and development. For instance, Mr. Kabelac's report and other reports indicate some notable innovations in dam-spillway-power unit layout and design, resulting in substantial benefits in construction economy and turbine efficiency.

Besides the more specialized or specifically-directed research of the various development and operating agencies, very extensive and coordinated programs of research and experimental development are carried on under the aegis of the general scientific agency, The Academy of Sciences of the U.S.S.R.[12] Perhaps a more effective environment for general, yet well-coordinated scientific developments can be attained through a combined use of the general science agency and the specialized agencies.

In the United States it would appear that we are not fulfilling our need for investments in scientific and technological research and for experimentation in the fields of water resources, river-system hydrology, hydraulics and river works, energy sources, power generation and transmission, energy utilization, water supply and pollution control, fish biology and engineering, recreational requirements, etc. Undoubtedly, handsome economic and social dividends could be achieved through a basic, well-coordinated and well-financed program of investment in this field.

Other World Approaches to River Development.—A good deal of what has been said previously about the desirability of being aware of river development in the U.S.S.R. has application to China. Our tendency now seems to be to discount the potentials of technical and physical progress in China, much as in the case of the U.S.S.R. a generation earlier. Apparently, accurate information on progress in China is harder to come by than that concerning the U.S.S.R., but various scattered reports do show gigantic efforts to bring China's rivers under control, and from source of sorrow into great usefulness. Examples are reports of planning and development on the unparalleled Yangtze Gorge project for hydroelectric power, flood control, irrigation, and other beneficial purposes, and on multiple-purpose control of the Huang Ho, including a long irrigation canal into the arid region of Inner Mongolia.

Naturally, we should not stop at this point in our observation of the development of plans for the world's great rivers. The emerging plans for the development of the Indus Vally and Rajasthan Desert by Pakistan and India and those for the Mekong in Southeast Asia, for instance, will warrant our attention both for the broad socio-economic concepts and the engineering features that inevitably will be brought out.

ERRATA

Journal of the Waterways and Harbors Division

Proceedings of the American Society of Civil Engineers

"Russian River Channel Works" by Israel H. Steinberg, November 1960, Proc. Paper 2647.

Beginning on p. 19 add figure captions as follows:

FIG. 1. GENERAL MAP OF RUSSIAN RIVER BASIN

FIG. 2. COMPLETED CHANNEL IMPROVEMENTS

FIG. 3. FLEXIBLE FENCE LOOKING UPSTREAM, TAKEN JUST AFTER COMPLETION (JAN. 1957)

FIG. 4. FLEXIBLE FENCE (OCT. 1959)

FIG. 5. CHANNEL TRAINING WORKS AT RIVER MILE 53.2 LOOKING UPSTREAM (JAN. 1957)

FIG. 6. CHANNEL TRAINING WORKS AT RIVER MILE 53.2 LOOKING UPSTREAM (OCT. 1959)

FIG. 7. CHANNEL TRAINING WORKS-RIGHTBANK, NEAR RIVER MILE 53.0 (OCT. 1959)

FIG. 8. CHANNEL TRAINING WORKS AT RIVER MILE 53.2. VIEW BEHIND MAIN JACK LINE NEAR MIDDLE OF INSTALLATION SHOWING EXTENSIVE SILT DEPOSITS (MARCH 1960)

FIG. 9. VIEW OF WORKS TAKEN FEB. 1958 DURING MODERATE FLOW

FIG. 10. CHANNEL BARRIER JACKS, RIVER MILE 52.9. VIEW FOLLOWING INSTALLATION (DEC. 1956)

FIG. 11. CHANNEL BARRIER JACKS, RIVER MILE 52.9 (SEPT. 1957)

FIG. 12. CHANNEL BARRIER JACKS, RIVER MILE 52.9. VIEW LOOKING UPSTREAM FOLLOWING PERFORMANCE DURING ONE SEASON AFTER RECONSTRUCTION (MAY 1958)

FIG. 13. SAME VIEW AS FIG. 12 (APRIL 1960)

FIG. 14. WIRE MESH AND GRAVEL BLANKET. RIVER MILE 52.2 (DEC. 1956)

FIG. 15. APPROXIMATELY SAME VIEW AS FIG. 14. (APRIL 1960)

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by (PP). For titles and other contents, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 32 (January 1962) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2703 is identified as 2703(ST1) which indicates that the paper is contained in the first issue of the Journal of the Structural Division during 1961.

VOLUME 86 (1960)

MAY: 2460(AT3), 2461(ST3), 2462(AT3), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA3), 2469(HY3), 2470(ST3), 2471(SA3), 2472(ST3), 2473(ST3), 2474(SA3), 2475(ST3), 2476(SA3), 2477(ST3), 2478 (HY3), 2479(SA3), 2480(ST3), 2481(SA3), 2482(CO3), 2483(CO3), 2484(HY3), 2485(HY3), 2486(AT3)[c], 2487 (CP1)[c], 2488(CO3)[c], 2489(HY3)[c], 2490(SA3)[c], 2491(ST3)[c], 2492(CP1), 2493(CO3).

JUNE: 2494(HY3), 2495(HY3), 2496(ST3), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(EM3), 2502 (EM3), 2503(PO3), 2504(WW3), 2505(EM3), 2506(HY3), 2507(WW3), 2508(PO3), 2509(ST3), 2510(EM3), 2511 (EM3), 2512(ST3), 2513(HW3), 2514(HY3), 2515(PO3), 2516(EM3), 2517(WW3), 2518(WW3), 2519(EM3), 2520 (PO3), 2521(HY3), 2522(EM3), 2523(ST3), 2524(HY3), 2525(HY3), 2526(HY3), 2527(EM3), 2528(ST3), 2529 (HW3), 2530(IR3), 2531(HY3), 2532(EM3)[c], 2533(HW3)[c], 2534(WW3), 2535(HY3)[c], 2536(IR3)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST3)[c], 2540(WW3)[c].

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AUGUST: 2564(SM4), 2565(IR), 2566(ST8), 2567(EM4), 2568(PO4), 2569(PO4), 2570(HY8), 2571(EM4), 2572(EM4), 2573(EM4), 2574(SM4), 2575(EM4), 2576(SM4), 2577(HY8), 2578(EM4), 2579(PO4), 2580 (EM4), 2581(ST8), 2582(ST8), 2583(EM4)[c], 2584(PO4)[c], 2585(ST8)[c], 2586(SM4)[c], 2587(HY8)[c].

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OCTOBER: 2615(EM5), 2616(EM5), 2617(ST10), 2618(EM5), 2619(EM5), 2620(EM5), 2621(ST10), 2622(EM5), 2623(EM5), 2624(EM5), 2625(EM5), 2626(EM5), 2627(EM5), 2628(EM5), 2629(ST10), 2630(ST10), 2631(PO5)[c], 2632(EM5)[c], 2633(ST10), 2634(ST10), 2635(ST10)[c], 2636(SM5)[c].

NOVEMBER: 2637(ST11), 2638(ST11), 2639(CO3), 2640(ST11), 2641(SA6), 2642(WW4), 2643(ST11), 2644(HY9), 2645(ST11), 2646(HY9), 2647(HW4), 2648(WW4), 2649(WW4), 2650(ST11), 2651(CO3), 2652(HY9), 2653(HY9), 2654(ST11), 2655(HY9), 2656(HY9), 2657(SA6), 2658(WW4), 2659(WW4)[c], 2660(SA6), 2661(CO3), 2662(CO3), 2663(SA6), 2664(CO3)[c], 2665(ST9)[c], 2666(SA6)[c], 2667(ST11)[c].

DECEMBER: 2668(ST12), 2669(IR4), 2670(SM5), 2671(IR4), 2672(IR4), 2673(IR4), 2674(ST12), 2675 (EM6), 2676(IR4), 2677(IR4), 2678(ST12), 2679(EM6), 2680(ST12), 2681(SM5), 2682(IR4), 2683(SM5), 2684(SM5), 2685(IR4), 2686(EM6), 2687(EM6), 2688(EM6), 2689(EM6), 2690(EM6), 2691(EM6)[c], 2692 (ST12), 2693(ST12), 2694(HW5)[c], 2695(IR4)[c], 2696(SM5)[c], 2697(ST12)[c].

VOLUME 87 (1961)

JANUARY: 2698(PP1), 2699(PP1), 2700(HY1), 2701(SA1), 2702(SU1), 2703(ST1), 2704(ST1), 2705(SU1), 2706 (HY1), 2707(HY1), 2708(ST1), 2709(PO1), 2710(HY1), 2711(HY1), 2712(ST1), 2713(HY1), 2714(PO1), 2715 (ST1), 2716(HY1), 2717(SA1), 2718(SA1), 2719(SU1)[c], 2720(SA1)[c], 2721(ST1), 2722(PP1)[c], 2723(PO1)[c], 2724 (HY1)[c], 2725(ST1)[c].

FEBRUARY: 2726(WW1), 2727(EM1), 2728(EM1), 2729(EM1), 2730(WW1), 2731(EM1), 2732(EM1), 2733(WW1), 2734(EM1), 2735(EM1), 2736(EM1), 2737(PL1), 2738(PL1), 2739(PL1), 2740(PL1), 2741(EM1), 2742(ST2), 2743(EM1), 2744(WW1), 2745(WW1), 2746(EM1), 2747(WW1), 2748(EM1), 2749(WW1), 2750(WW1)[c], 2751 (EM1)[c], 2752(EM1)[c], 2753(PL1)[c], 2754(ST2)[c], 2755(PL1).

MARCH: 2756(HY2), 2757(IR1), 2758(AT1), 2759(CO1), 2760(HY2), 2761(IR1), 2762(IR1), 2763(HY2), 2764 (ST2), 2765(HY2), 2766(HW1), 2767(SA2), 2768(CO1), 2769(IR1), 2770(HY2), 2771(SA2), 2772(HY2), 2773 (CO1), 2774(AT1), 2775(IR1), 2776(HY2), 2777(HY2), 2778(SA2), 2779(ST2), 2780(HY2), 2781(HY2)[c], 2782 (HW1)[c], 2783(SA2)[c], 2784(CO1), 2785(CO1)[c], 2786(IR1)[c], 2787(ST2)[c], 2788(AT1)[c], 2789(HW1).

APRIL: 2790(EM2), 2791(EM2), 2792(SM2), 2793(EM2), 2794(EM2), 2795(SM2), 2796(SM2), 2797(EM2), 2798 (EM2), 2799(EM2), 2800(EM2), 2801(EM2), 2802(ST4), 2803(EM2)[c], 2804(SM2)[c], 2805(ST4)[c].

MAY: 2806(SA3), 2807(WW2), 2808(HY3), 2809(WW2), 2810(HY3), 2811(WW2), 2812(HY3), 2813(WW2), 2814(HY3), 2815(WW2), 2816(HY3), 2817(HY3), 2818(SA3), 2819(WW2), 2820(SA3), 2821(WW2), 2822 (WW2)[c], 2823(HY3), 2824(SA3), 2825(HY3), 2826(SA3)[c], 2827(HY3)[c].

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c. Discussion of several papers, grouped by divisions.

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